ENGINEERING PROPERTIES OF SOME MUSKEGS RELATIVE TO ROAD CONSTRUCTION

K. O. ANDERSON

OCTOBER 1959

THE UNIVERSITY OF ALBERTA

ENGINEERING PROPERTIES OF SOME MUSKEGS RELATIVE TO ROAD CONSTRUCTION

A DISSERTATION

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

by

K. O. ANDERSON, B.Sc.

EDMONTON, ALBERTA

OCTOBER, 1959

ACKNOWLEDGEMENTS

Acknowledgements are made to Dr. R. M. Hardy, under whose direction this study was conducted, and to Professor S. R. Sinclair, for the many encouraging discussions during this investigation.

Acknowledgement is made to the Production Research and Technical Service Department of Imperial Oil Limited, Western Region, who sponsored the construction program as well as the preliminary investigations and supervision during the summer program of field work. Special indebtedness is afforded to Mr. R. A. Hemstock of Imperial Oil, whose planning and foresight brought this research project into being.

The assistance of many other personnel of Imperial Oil and their valuable assistance in securing field data is also gratefully appreciated and acknowledged. The personal interest of Mr. John Staples of McGregor Johanson Limited greatly assisted in the smooth operation of the construction details, and is also acknowledged.

Acknowledgement is made to Dr. N. W. Radforth, whose comments regarding the classification of muskeg in this test area have helped the author in achieving a better understanding of the various facets of this subject.

SYNOPSIS

Exploration and development of vast areas of Canada involves the construction of various types of roads on muskeg.

In order to obtain information on some road construction techniques and to relate the behaviour of earth fills on muskeg to the engineering properties of this organic material, several test fills were constructed during the summer of 1958. This test program was undertaken by the Production Research Department of Imperial Oil of Canada and the Civil Engineering Department of the University of Alberta.

Results obtained showed a close correlation between the shearing strength of the muskeg as measured with a vane shear apparatus and the shearing resistance as computed from a fill loaded to failure. Further relationships between the moisture content and the shearing strength of two classifications of muskeg were obtained. Information on magnitudes and rates of settlement of fills on muskeg were also obtained.

Investigation on the effect of plastic and asphalt fibreglass membranes to prevent the movement of moisture into road fills constructed on muskeg was also carried out. The membranes used were unsuccessful for this purpose.

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER I

INTRODUCTORY CHAPTER

With the continuing development of our natural resources in Canada, particularly in the northern areas, the problems encountered with muskeg occurrences are becoming increasingly important. Soil surveyors of the Federal Department of Agriculture have estimated that muskeg, or organic terrain as it is sometimes called, covers some 12 percent of the entire country, roughly 435,000 square miles.[1*] Exploration and development of these areas involves fundamental transportation problems, including the construction of roads of varying standards.

In order to advance past work involving muskeg problems, and to obtain immediate information on some road construction techniques, the Production Research and Technical Service Department of Imperial Oil Limited, Western Region, and the Civil Engineering Department of the University of Alberta conducted a program of construction of various test fills on muskeg during the summer of 1958. The information obtained during the construction and observation of these test fills will comprise the major portion of this study. While this was primarily a field project, a limited amount of laboratory investigation was done in order to supplement the field data.

---

* THE SUPERSCRIPTS REFER TO BIBLIOGRAPHY REFERENCES

Two specific problems were involved in this study and could be stated as follows:

(1) To determine whether the shearing strengths as measured by a vane shear test apparatus could be used in stability computations of fills constructed on muskeg, and

(2) To investigate the effect of plastic and asphalt fibreglass membranes in preventing the movement of moisture into road fills constructed on muskeg.

Related problems, although considered to be of lesser importance in this study, were the variations in shearing strength and moisture content of muskeg of different classification, as well as the magnitudes and rates of settlement of fills constructed on muskeg.

Since the purpose for which a road is to be used governs the standard to which it is to be built, the construction of roads for exploration and development can be classified as two main types, namely,

(1) minimum standard -- temporary access type of road

(2) higher standard -- heavier and more permanent type of road.

In the minimum standard type of road the concern is mainly that of carrying a limited amount of traffic for a relatively short period of time. If the imported fill

material could be prevented from taking on additional moisture from the supporting muskeg a much lighter fill than otherwise required could be used. In addition to this the spring breakup problem associated with excessive frost action would not be as severe as does normally occur since the major source of water would be cut off.

In the higher standard type of road where more permanence is required a heavier fill is usually built in order to maintain the road surface some distance above the surrounding water level. When this is done the problems of shearing failure in the supporting material together with the amount and rate of settlement of the fill are of main concern.

With these problems in mind a program which consisted of the construction of several light earth fills on membranes placed directly on the muskeg surface, and a fill loaded to failure was undertaken. Construction equipment was arranged for by Imperial Oil Limited who also provided the needed survey crew and assistance during certain phases of construction. Preliminary testing, layout, instrumentation and observation during construction was handled by the writer.

The several phases of this investigation are dealt with in the remainder of this thesis. Chapter II gives a

general description of the test site and layout. Chapter III describes the vane shear apparatus and the preliminary testing on the site. Chapter IV outlines the construction and analysis of the fill loaded to failure, relating the measured shearing strength to the computed shearing resistance of the muskeg. Chapter V deals with the use of membranes on the light fill sections. Chapter VI considers the problem of settlement of fills on muskeg. Chapter VII gives a summary of shear strengths of muskeg as found in this study together with results of previous investigators. The final Chapter VIII states the conclusions and recommendations resulting from this study.

This investigation is the third one conducted through the Civil Engineering Department of the University of Alberta. The first such thesis, by Captain S. Thomson of the Royal Canadian Engineers, entitled "An Investigation Into the Engineering Properties of Muskeg" in 1955 reported on the development and use of a vane shear apparatus, together with routine soil mechanics tests conducted on a muskeg at Mile 253 along the Alaska Highway.[2] The second thesis, by Captain R. C. Wyld also of the Royal Canadian Engineers, entitled "A Further Investigation Into The Engineering Properties of Muskeg" in 1956 described a modified vane shear apparatus which used a torque wrench

instead of a cable tensioned spring, and gave results of similar tests on various muskegs in the Pembina Oil Fields.[3] This present thesis is purposed to advance and broaden the knowledge of the engineering behaviour of muskeg.

CHAPTER II

TEST SITE

In order to find a suitable test site a survey of known muskeg deposits in the general Edmonton area was undertaken. A uniform deposit, twelve feet in depth or more, located in such an area that a road could be built to interconnect wellsites and thereby test traffic could easily be provided, was what was desired. After inspecting various locations, it was realized that a compromise would have to be made in order to locate a deposit having a depth of twelve feet. It was then decided that the road could not be located to have through traffic operate on it.

A suitable test site was then located in the NW 1/4-33-48-4-W5 approximately seventy miles southwest of Edmonton. The surface elevation as obtained from a topographic map was 2775 feet above sea level. Topographic conditions defined the muskeg area as being of a closed pond form extending about one mile in length and one-half mile in width. Being undrained, the water level was at the surface of the muskeg. Utilizing the Radforth Classification System[1] of classifying the vegetation cover present, field investigation showed that the Classes F and I were the most

prevalent with varying amounts of B, D and E. The descriptive information for the nine pure coverage classes is given in Appendix I. Cover formulae changed from AEI at the edge of the muskeg area, to BEI, BDF and FI progressively towards the center of the deposit. The central area was predominantly FI with ridges running parallel to each other at intervals of fifty to eighty feet. These ridges consisted of BEI with patches of localized BDI between five and ten feet in width, the surface elevation being approximately one-half foot above the surrounding FI. Figure 1 is an aerial photograph of this muskeg deposit, while Figure 2 shows a view of the terrain from the ground. The depth of muskeg generally ranged from ten to twelve feet with the maximum observed depth being fourteen feet. The sub-surface material, some times referred to as peat, could be described as fine fibrous ranging to amorphous. Since peat can most readily be described by the extent to which wood and fibres are present, the above description indicates the lack of coarse woody particles. Except for the first three or four feet below the ridges of BEI in which some coarse fibres and small roots were observed, the sub-surface constitution was surprisingly homogeneous throughout the test area, as determined by visual identification. The bottom several feet of the muskeg

deposit, particularly the bottom two feet was well decomposed with no fibrous appearance. Samples of the organic sub-surface materials were taken and forwarded to Dr. Radforth for more detailed analysis. The mineral soil underlying this deposit was a blue silty clay of medium to high plasticity.

The location of the test road was in the extreme south-west corner of the quarter section. Referring to Figure 3, the road was located along the centerline of a one hundred foot wide right-of-way which ran north, parallel and immediately east of an unused road allowance, for a distance of approximately seven hundred feet. At this point the direction of the line changed to a northeasterly direction and continued for a further 450 feet. The southern end of this right-of-way abutted the provincial highway which passes by on the south of this muskeg deposit. Progressing from the southern end of the right-of-way adjacent to the highway in a northerly direction, the vegetation cover formulae could be described as AEI, BEI, BDF, patches of EI amidst FI, FI and successive ridges of BEI.

The test layout was planned such that the light fill sections consisting of a control section and two membrane sections, each 125 feet in length, would begin at a point

approximately five hundred feet north of the highway. At this point the depth of muskeg was seven feet with the surface cover being AEI. The control section was constructed in the same manner as the next two sections except that no membrane was placed on the muskeg before spreading the earth fill. The fill to failure section was located on FI muskeg between two ridges of BEI beyond the light fill sections. The depth to mineral soil in this area was twelve feet.

CHAPTER III

VANE SHEAR APPARATUS AND TEST DATA

The vane shear test apparatus consisted of a four bladed vane which was forced into the soil to be tested, with the torque required to rotate the vane and thus shear a cylinder of soil being measured by a torque wrench. The vane shear test apparatus used was that developed jointly by Wyld and Walsh,[3] with some modifications. The vane apparatus assembled in a position for testing is shown diagrammatically in Figure 4.

The main change to the apparatus was the replacement of the 5/8 inch diameter pony drill rods with standard XRT diamond drill rod. The outside diameter of this drill rod was 1.094 inches, but being hollow weighed only 1.33 pounds per foot as compared to 1.6 pounds per foot for the pony drill rod. Due to a larger polar moment of inertia for the XRT drill rod, the angular rotation or twist under an applied torque would be only 1/6.1 times that for an equal length of pony drill rod.

The advantage of using this drill rod was checked experimentally and is shown in Figure 5. With the vane plus attached three foot length of pony rod, the total twist for a torque of sixty foot pounds was observed to be ten degrees.

For the same torque, but with an additional ten feet of new XRT drill rod, the total twist between the extreme ends of the rod and vane was thirteen degrees. This torque corresponds to a shearing strength of three hundred pounds per square foot when using a vane of the given dimensions, a value higher than that observed for the muskeg tested. This shows that for the depths and shearing strengths measured the twist in the drill rod and vane assembly was considerably less than thirteen degrees.

Minor modifications also include a new thrust bearing at the top of the casing, a graduated circle for observing angular rotation, and a quick acting clamp for holding the casing and test apparatus at any particular depth.

It is generally considered that the vane shear apparatus measures the undrained shearing strengths of saturated fine-grained soils, and that these results are comparable to shearing strengths obtained from unconfined compression tests.[4] While some recent investigations have been made with the vane shear apparatus on soils having an angle of internal friction, the major work to date has been with saturated clays.[5] For saturated clays which are brought to failure without drainage, the Coulomb equation for shearing resistance of the soil -

$$S = C + p \tan \phi$$

reduces to $S = C$ where S = shearing resistance

C = cohesion

p = intergranular pressure

ϕ = angle of internal friction

This latter relationship is assumed to apply in the case of muskeg material.

Appendix 2 contains a development of the theory for the vane shear apparatus. Figure 6 is a conversion chart relating the shearing strength to the applied torque.

The procedure followed in conducting a test was as follows:

1. The vane apparatus and casing was assembled.
2. The vane was advanced to the desired depth by standing and applying weight on the casing clamp set at the proper position. If the root structure or material was too resistant for this method, a hand auger hole was put down to within one-half vane length of the desired depth. The vane assembly was then advanced to the full depth of the blades.
3. The graduated circle was attached to the upper end of the casing by means of the threaded coupling.
4. The torque wrench was connected to the drill rod and

the circle then positioned for zero reading.

5. The torque was then applied at a rate such that the angular rotation was thirty degrees per minute. Record was taken of the torque at each fifteen degree interval until the maximum reading had been obtained.
6. The vane was then rotated quickly four complete revolutions and the remoulded strength then obtained in the same manner starting one minute after the completion of the fourth revolution.
7. Required lengths of drill rod were added and the assembly advance to the next depth. Steps 3 to 6 were then repeated. While this rate of rotation was quite high, taking about three or four minutes to complete one strength determination, this rate was the slowest at which the torque could be steadily applied. This rate was the same as that arrived at by Wyld in his work.

Auger holes were put down about two feet away from where the vane tests were run in order to obtain samples for moisture and ash contents. Three inch diameter shelby tube samples were taken at the top and bottom of the deposit for laboratory testing. In all cases the recovered tube samples were greatly compressed.

The measured shearing strengths of the FI muskeg

followed the previously reported trend of increasing with depth. Relatively consistent results were obtained on all FI muskeg tested. While not showing as consistent results the BEI muskeg showed a loss in strength down to a depth of five to eight feet, then an increase with greater depth. The moisture profiles with depth followed an inverse ratio; that is in the case of the FI muskeg the moisture content decreased going from the one foot to three foot depth, increased to the five foot depth, and then decreased again to the eleven foot depth. The trend in the BEI muskeg was to an increase in moisture content from the surface to the five to eight foot depth, then a decrease with greater depth. These results are summarized in Figure 7.

The moisture content was defined as the loss in weight, expressed as a percentage of the dry material, after the original sample had been dried for twenty-four hours at 110 degrees Centigrade.

A plot of moisture content versus undisturbed shearing strength as shown in Figure 8 illustrates a direct variation from about 250 pounds per square foot at 700 per cent moisture content down to 100 pounds per square foot at 1400 per cent. A band of plus or minus 50 pounds per square foot from a straight line joining these two points would encompass nearly 90 per cent of the test results.

Difficulty was experienced in obtaining representative samples for moisture content determinations particularly at the higher moisture contents. This fact would account for much of the scatter of this plot. Samples were taken from the side of the auger hole at shallow depths and from the bottom of the auger at other depths. Care was taken not to compress the sample material any more than necessary.

A similar plot of remoulded shearing strengths versus moisture content showed only slight variation with moisture content. All values were below 100 pounds per square foot. The sensitivity ratio of undisturbed to remoulded shearing strength ranged from a low of 1.5 to a high of 3.7.

The ash content as determined by the weight of residue after drying at 300 degrees Centigrade divided by the weight of dry sample after twenty-four hours at 110 degrees Centigrade, varied between a rather narrow range of 10 to 25 per cent.

As has been noted earlier the sub-surface constitution of the FI and BEI muskeg appeared to be relatively homogeneous, except for the first three or four feet in depth. A review of the limited number of tests on the BEI shows consistently higher shearing strengths than the FI down to a depth of about eight feet. At this depth there was no significant difference in measured shearing

strengths. It would appear from a review of this data that the higher strength under the BEI would be due to the lower moisture content rather than to a reinforcing effect of any root structure. This lower moisture content could in turn be due to the transpiration of moisture by the larger surface growth.

Atterberg limit tests were attempted on two samples obtained from the shelby tubes in order to determine their usefulness as physical property tests for this organic material. A sample of well decomposed material taken from the eleven foot depth had a liquid limit of 505 per cent and a plastic limit of 470 per cent. The ash content was approximately 23 per cent, indicating the low mineral soil content. The sample taken from near the surface of the muskeg was too fibrous to run the limit tests on.

While Wyld was able to relate ash content to plasticity index on a limited number of tests, it is felt that the Atterberg limit tests can only be run on well decomposed non-fibrous materials and therefore are of limited usefulness for muskeg investigations.

CHAPTER IV

FILL TO FAILURE CONSTRUCTION AND ANALYSIS

1. CONSTRUCTION

The fill to failure section was built on an area of FI muskeg which was twelve feet in depth with the water level at the surface. While it would have been preferable to have a greater depth than this in order to ensure that the maximum shearing stresses would remain in the muskeg material for a practical width of embankment, no other suitable test site was available. Estimating that a height of ten feet of fill might be necessary, the base width of forty feet was chosen since the minimum top width was considered to be twenty-four feet.

Settlement platforms consisting of two-foot square pieces of plywood with one inch diameter pipe rods were placed along centerline and fifteen feet to the left and right of centerline at twenty-five foot intervals. The purpose of these platforms was to show the amount of settlement of the fill during construction, and to indicate when failure had occurred.

Piezometers of the porous stone tube type were installed in order to measure the hydrostatic pressures during and following the construction of the fill. A

schematic sketch of the piezometer installation is shown in Figure 9.

Two installations were made along the centerline and two were set fifteen feet to the right of this line. They were set at the six to eight foot and ten to twelve foot depths at each line, in order to observe any possible variation of pore water pressure with depth. Installation procedure differed from that as recommended by Casagrande[6] in the manner of lowering the tube below water level. The prescribed manner was to keep a positive head on the water inside the tube so that there would be a flow of water out of the porous tube when being lowered into position. With the absence of a portable pump or sufficiently large reservoir to do this, this procedure was not followed. Care was taken in lowering the tube so that organic material did not come in contact with the porous stone. Success of this method of installation was indicated by the almost instantaneous rise and fall in the water level in the plastic stand pipe when construction equipment came within the near vicinity of the installation. It is significant to note that this procedure was very similar to that now recommended by Casagrande, as described in a revision of procedures prepared in 1958.

As a means to assist in locating the plane of

failure a method similar to that described in reference (7) was used. This consisted of several flexible plastic tubes of one-half inch inside diameter driven into the muskeg down to the mineral soil layer. The curvature of each tube in place was checked by lowering successive lengths of one-quarter inch rod ranging from six to twenty-four inches in length in six inch increments. The lower end of each tube was sealed with a tight fitting plug which also served as a driving point for a one inch diameter pipe slipped over the tube. When the required depth had been reached the pipe was raised leaving the plastic tube in position.

Several lines of guide stakes on each side of the test fill base were set in order to detect lateral movements during construction. These stakes were on lines extending radially outward from an observation platform situated on centerline about three hundred feet away, facilitating observations with a transit from one location. A plan of this layout is shown in Figure 10, with photographs of the site in Figures 11 and 12.

Construction of the fill was carried out in the same way as the previously built membrane sections, that is, hauling out fill material by truck and spreading with a D-6 dozer. The first spread of about five feet of fill had

been placed over the first third of the test area when rain stopped further hauling for one week. When construction was resumed fill was placed for two additional days. The height of fill reached a maximum of 7.0 feet above water level before a small crack appeared along the centerline of the fill. A drop of 0.7 feet occurred overnight, indicating a shearing failure of the supporting material. Settlement along centerline of the fill before failure was approximately 6.0 feet. The maximum excess hydrostatic pressure on centerline at a depth of ten to twelve feet below the original muskeg surface was equivalent to a head of 8.00 feet of water. Maximum lateral movement of the side stakes was 0.5 feet before failure, with no noticeable rise in surface elevation. Figure 13 shows a plot of the elevations of the top of fill, bottom of fill, and piezometer readings during the time of loading.

And additional three feet of material placed three days after the time of initial failure did not succeed in causing more pronounced movements. It was hoped that this would help to locate the toe of the failure surface. Unfortunately, during the placement of this extra fill, the one remaining piezometer along the centerline became buried. Because of this the time required for dissipation of excess hydrostatic pressures could not be determined.

The material used in constructing the fill was a silty sand of low plasticity. The natural moisture content of this material was quite low, ranging up to a maximum of 10 per cent. Density tests taken on the fill after construction indicated a wet unit weight of 126 pounds per cubic foot and a dry unit weight of 115 pounds per cubic foot.

II <u>FAILURE ANALYSIS</u>

As noted previously, one of the major concerns of this study was to determine whether the shearing strengths as measured by the vane shear apparatus could be used for stability analysis of fills constructed on muskeg. Since the fill loaded to failure was in effect a large scale shear test, comparisons of the computed shearing resistance at the time of failure could then be made with the measured vane shearing strengths in order to determine their reliability.

In computing the shearing resistance, several methods of analyses were tried. Each method was limited by certain qualifying assumptions which may or may not have been satisfied by the actual conditions. By eliminating the methods of analyses that were not applicable, the most appropriate method was then considered as giving the computed value of the shearing resistance of the muskeg.

The methods used were:

1. Circular Arc Analysis
2. Computation of Stresses by Theory of Elasticity
3. Bearing Capacity using Plastic Equilibrium Theory
4. Sliding Block Analysis

Each of the methods investigated will be discussed in turn.

In calculating the load on the muskeg, account was taken of the different unit weights due to the varying water conditions. The fill material when placed was assumed to have a total unit weight of 120 pounds per cubic foot. In a zone extending 1.5 feet above the water level, the material became wet by capillary rise of moisture, and therefore was assumed to have a unit weight of 130 pounds per cubic foot. Below the water level the submerged unit weight was assumed to be 60 pounds per cubic foot. The submerged unit weight of the muskeg was considered to be 5 pounds per cubic foot, a reasonable value for the observed moisture contents.

1. <u>Circular Arc Analysis</u>

The failure surface was assumed to be cylindrical in shape, thus appearing on a cross section as a circular arc. Referring to Figure 14, the circular failure arc was assumed to pass through a point on the bottom of the fill near the centerline and to touch the top of the mineral soil

layer. The location of the first point was based on observed settlement platform movements, while the higher shearing strengths of the underlying mineral soil layer established the lowest point. The location of the center of this failure circle, known as the critical circle, was found by a trial and error method. On this critical circle, considering the weight of that portion of fill tending to cause rotation as the driving force, and the weight of that portion of the fill tending to resist rotation together with the shearing resistance along the arc as the resisting force, the ratio of the resisting moment to the driving moment about the center of rotation must be a minimum. Having located the critical circle and considering a one-foot slice of the cross section of the fill, the actual computation for determining the average shearing resistance along the arc was made by using the following formula:

$$S = \frac{W_1 d_1 - W_2 d_2}{l \cdot r} \qquad \text{(Ref. 8, p 183.)}$$

in which

S - average shearing resistance, pounds per square foot.

W_1- weight of slice tending to produce rotation,) pounds

W_2- weight of slice tending to resist rotation,

d_1 and d_2 - moment arms of W_1 and W_2 respectively about center of rotation, feet.

l - length of critical failure arc, feet.

r - radius of critical failure arc, feet.

The average shearing resistance thus computed would be the minimum strength required for equilibrium.

In addition the following assumptions were made:

1. The length of arc did not include that portion in the fill.
2. Active earth pressures on the surface AC through the fill were neglected.
3. Weights of muskeg were not included in moment computations.
4. The shearing resistance of the muskeg was assumed to be purely cohesive, that is $\phi = 0$.
5. Excess water pressures on the surface AC were neglected.
6. Excess water pressures along the failure arc AB were not included in the computations.

Assumption 1 was made since the extreme differential settlement between the center and sides of the fill would have placed tension stresses in the lower part of the fill, thereby likely causing tension cracks to occur. The way in which a crack appeared along the centerline of the fill, and subsequent settlement observations seems to justify this assumption. For the same reasons, assumption 2 would also appear to be valid. Computations indicated that the change in average shearing resistance by making assumption 3 amounted

to a maximum of 5 pounds per square foot, well within the limits of test accuracy. Assumption 4 was made on the basis of reports of other investigators that the strength of well decomposed peat or muskeg could be considered as cohesive in nature. This is further justified by the indications from the piezometer readings that very little drainage took place during the time of loading. If the excess water pressures observed in the muskeg also occurred along surface AC, assumption 5 would introduce some error, but not over 10 per cent, in the computed shearing resistance. Assumption 6 was justified on the basis of the earlier assumption of the muskeg being cohesive and possessing no internal friction.

This analysis gave a computed average shearing resistance of 160 pounds per square foot.

2. Computation of Stresses by Theory of Elasticity

This method of analysis is a theoretical method involving the theory of elasticity and serves primarily to furnish a picture of the distribution of shearing stresses below a loaded area. Figure 15 shows the distribution of shearing stresses below the constructed fill, based on charts prepared by Leo Jurgenson[9] giving a graphical application of the Boussinesq equations. These stresses are based on the assumption that the theory of elasticity applies in this case, that the supporting material is homogeneous and isotropic, and of infinite extent.

Since failure did occur in the supporting material,

there must have been a zone in which there was plastic flow instead of elastic deformation. As well, the depth of muskeg was only twelve feet, hence the assumption of a homogeneous and isotropic soil is not satisfied.

Despite the admitted deficiencies of this method, there is merit in considering what shearing stresses would be induced if the qualifying assumptions were satisfied.

In this analysis it was assumed that the load was applied at the bottom of the fill at a uniform depth of six feet below the surface of the muskeg. The submerged weight of muskeg adjacent to the fill and above this depth, amounted to approximately 30 pounds per square foot and therefore was neglected when considering the fill load of 1220 pounds per square foot. This latter load was computed using the height of fill at the time of failure and the respective unit weights. This was 5.5 feet at 120 pounds per cubic foot, 1.5 feet at 130 pounds per cubic foot, and 6.0 feet at 60 pounds per cubic foot.

The shearing stresses for various points along the critical arc as located in the circular arc analysis were computed and plotted against their distance along the arc. Since the larger stresses must have caused the material to yield plastically, thereby transmitting part of the stress to adjoining material, there would have been a change in the

stress pattern from that based on elastic deformations. This progressive failure would tend to evenly distribute the stresses until the total load could be resisted or until complete failure occurred. For this reason the average shearing stress along the critical arc was determined by summing the computed stresses according to Simpson's rule and dividing by the length of arc.

The average shearing stress so determined was 150 pounds per square foot.

3. Bearing Capacity using Plastic Equilibrium Theory

Another possible way to analyze this failure section would be to consider the fill as a continuous shallow footing. Reference 8, page 167 considers mathematical investigations concerning the state of plastic equilibrium beneath continuous footings. For the case of a shallow footing having a rough base, on a cohesive soil for which $\emptyset = 0$, the ultimate bearing capacity is given by the formula:

$$q_d = 5.70\ c$$

where

q_d = load intensity at which sinking of the footing occurs

c = cohesion of the soil.

For this particular case, at the time of failure, the load intensity was 1220 pounds per square foot. Assuming that the conditions necessary for the application of this formula

were satisfied, the required cohesion c for equilibrium would be 210 pounds per square foot.

Figure 16 is a diagram showing the idealized conditions for general shear failure of the material supporting the fill, considering the state of plastic equilibrium. This would be the failure pattern for which the above formula would be applicable.

It can be seen that the zones of radial shear extend well down into the mineral soil layer below the muskeg. Since the mineral soil has a much higher shearing strength than the muskeg, there is an abrupt discontinuity in the supporting soil. This would mean that the zones of plastic equilibrium could not extend down into the mineral soil.

It can also be seen that the wedge-shaped zone beneath the loaded area, which is in a state of elastic equilibrium, extends down into the mineral soil as well. This would mean that the fill is supported not only by the passive earth pressure of the muskeg, but partially by the underlying mineral soil through the action of this wedge-shaped zone.

For these reasons it can be stated that the physical conditions of this particular fill do not permit the use of the bearing capacity formula. It would appear that this method of analysis could be applicable only where the depth of muskeg was approximately 0.7 times the base width of the fill or greater.

4. Sliding Block Analysis

In this method of analysis it is assumed that a block of soil slides bodily along an approximately horizontal plane of weakness. Referring to Figure 17, the forces acting on the block of soil are shown. The forces tending to cause movement along plane AD are due to the resultant active earth and water pressures acting on the plane AB. The resisting forces are due to the resultant passive earth and water pressures on the plane CD together with the shearing resistance along the sliding surface AD.

For this analysis the following assumptions in addition to those mentioned previously regarding the unit weights of the fill and muskeg were:

1. The earth fill was cohesionless with $\phi = 30$ degrees.
2. The muskeg was purely cohesive with $\phi = 0$ degrees.
3. The sliding surface was horizontal, 20.0 feet in length, and at a depth of 6.0 feet below the muskeg surface.
4. Active and passive earth pressures were assumed to act horizontally.

Assumption 1 was made in keeping with the observed condition of the silty sand fill material and estimates of the angle of internal friction ϕ, found in reference 8.

Computations pertaining to this analysis are also shown on Figure 17. The resultant active earth pressure was determined by first finding the unit earth pressures at various depths by applying the coefficient of active earth pressure $K = \frac{1 - \sin \phi}{1 + \sin \phi}$, to the unit effective vertical stresses.

The volume of the earth pressure diagram then was the resultant earth pressure force acting on the plane AB. The resultant passive earth pressure acting on the plane CD was expressed by the formula,

$$P_P = \frac{1}{2} w h^2 + 2 s_{av} h$$

where w = unit weight of the muskeg,
h = depth of plane CD below surface of the muskeg,
s_{av} = average unit shearing resistance of the muskeg, required for equilibrium.

In case 1 it was assumed that there were no excess water pressures acting on plane AB, hence the only actuating force was the resultant active earth pressure. For this condition the required shearing resistance of the muskeg was 95 pounds per square foot.

Since excess water pressures of 8.0 feet were observed in the piezometer set at the ten to twelve foot depth below the muskeg surface, the possibility of excess water pressures in the fill was considered in case II. It was assumed that a minute crack extending to within three feet of the top of the fill had developed sufficiently to allow water pressures to act in addition to the active earth pressures. For these conditions the required shearing resistance was 235 pounds per square foot.

In case III the same water conditions as in II were assumed, however the earth pressures were not considered to act. This would be the case if a large crack had formed.

The required shearing resistance under these conditions was 140 pounds per square foot.

It can be seen from these results that the stability of the fill according to this method of analysis is largely dependent upon the water pressures in the fill at the time of failure. Using the value for the shearing resistance of the muskeg obtained from the Circular Arc Analysis, it would appear that the fill would fail by sliding if the excess water pressures were as in case II, but would be stable if conditions were as in case I or III.

Conclusions From Failure Analyses

Since the actual surface of failure could not be located with any certainty, it cannot be definitely concluded which method of analysis should apply in this case. Both the Circular Arc and Sliding Block methods of analyses could be equally valid, with indications tending to favour one method over the other being inconclusive.

It is significant, though, that both methods of analyses gave results which were of the same relative magnitude. The average shearing resistance of the muskeg by the Sliding Block analysis ranged from 95 to 235 pounds per square foot, with the more probable range being from 140 to 235 pounds per square foot. The Circular Arc method gave a value of 160 pounds per square foot for the average shearing resistance, while the average shearing stress along the critical arc by theoretical computations was 150 pounds per square foot.

The measured vane shearing strengths of the FI muskeg before failure ranged from 125 to 225 pounds per square foot at the three and eleven foot depths respectively. The overall average shearing strength was 150 pounds per square foot. From this it can be seen that the vane shearing strengths are of the same order of magnitude as the computed average shearing resistances and therefore seem to be applicable to stability analyses.

To check on the possibility of changes in the shearing strength of the muskeg during the time of loading, an attempt was made to run vane tests below the fill after failure. The soft condition of the fill below the water level prevented advancing an auger hole through this material so instead a test was taken to the side of the fill seven days after loading to failure. In this one test the average shearing strength increased to 190 from the original 150 pounds per square foot. In view of this it could be concluded that no significant change had taken place in the shearing strength of the muskeg during the relatively short loading period.

Performance of Test Instrumentation

Difficulty was experienced in setting the piezometers at the shallower depths of six to eight feet. With insufficient embedded length of the casing in the very soft FI muskeg, fill placed near the casing caused excessive movement.

The piezometers placed at the deeper depths worked successfully with immediate response to additional loading.

After the initial failure the extra fill placed later covered the exposed ends of the plastic tubing and therefore readings could not be taken during the time that the excess water pressures were being dissipated.

The method of using an electric probing device for determining the water levels in the plastic tubes was most satisfactory.

The plastic tube slide surface indicators did not yield any conclusive results. All tubes except one remained sufficiently straight to permit lowering the indicator rods to the full depth of the tubes. The one exception was at the far end of the fill where a shallow local failure developed as the first part of the fill was being placed. Lateral movements in this case were one to two feet and could be readily seen. It is possible that the magnitudes of differential movements were not large enough to cause significant bending of the plastic tubes, or that their resistance to bending was sufficient to cause flow around the tubes.

The settlement platforms enabled the determination of the magnitudes of settlement at each location during various stages of construction. A heavier pipe than the one inch diameter used would have provided a more rugged installation in order to withstand damage by the construction equipment, although this would not be necessary if care were taken in placing the fill around the pipe.

CHAPTER V

MEMBRANE SECTIONS

In order to investigate the possibility of improving the performance of the light fill or temporary access type of road by the use of membranes to prevent or reduce the movement of moisture into the imported fill, several test sections were built. The original plan was to construct a control section, a plastic membrane section, and an asphalt fibreglass membrane section, each being 125 feet in length. During construction this plan was changed somewhat, with the original sections being shortened and two sections being added, as is later described.

All growth larger than two feet in height was cut off close to the surface in order to make it possible to lay the membranes directly on the muskeg. The light woody and non-woody growth up to two feet in height was left in place, since it would not be practical to remove this in a large scale operation. The surface cover formulae varied from a short section of AEI at the start of the control section, to extensive areas of FI, with intermittent ridges of BEI.

Settlement platforms were set up on centerline at the midpoint of each of the three sections, with

piezometers in the same location on the control and plastic membrane sections only.

The roadway was to be built to a minimum top width of eighteen feet with as light a fill as possible to carry the loaded hauling equipment. During construction it was found that the width had to be increased, particularly in the vicinity of the settlement platforms and piezometers, so the final top width varied from twenty-two to twenty-six feet. As progressive settlement took place, additional fill material was hauled and placed. The quantity used for each section was recorded, with separate count made of the quantity used initially and the subsequent quantity needed to maintain traffic. Quantities were determined both by truck volume measurement and by borrow pit surveys.

Two trucks, a 10 cubic yard tandem axle and a 6 cubic yard single axle, were used for hauling with a D-6 tractor and dozer being used to spread the fill out over the muskeg.

<u>Control Section</u>

The control section was constructed in the same manner as the following membrane sections, except that the earth fill was placed directly on the muskeg. The purpose of this section was to provide a standard for later comparisons. Figure 18 shows a view of this section just after construction.

Plastic Membrane Section

The plastic membrane consisted of four mil thick polyethylene plastic, which came in long strips twelve feet in width. These strips had been fabricated from two six-foot wide strips. Lengths of thirty to thirty-two feet were cut and laid in a transverse direction to the center-line of the road. The edges of each strip were welded together in the field with the use of an electric heating unit of a type specifically used for this purpose, as shown in Figure 19. Fill was then hauled and spread over the plastic membrane, leaving a three to four foot width on each side uncovered so that these could be turned up. (Figure 20).

In following this procedure several difficulties were encountered, the major ones being:

1. In the field welding operation it was very difficult to adjust the amount of heat correctly so that a strong joint could be obtained without danger of overheating and melting the plastic.
2. Sharp roots and woody growth punctured the plastic in several places, especially when the fill placed the membrane in tension.
3. Several field welded joints pulled apart when the fill was being pushed out with the dozer. (Figure 21).

In view of this experience is concluded that the

plastic was unsuccessful as a membrane when placed directly on the muskeg.

Asphalt-impregnated Fibreglass Membrane Section

This membrane was similar to ones that have been used to a limited extent to waterproof basement walls, canal linings and other like structures. It combines the tensile strength of glass fibre mats with the waterproofing characteristics of asphalt.

This membrane was fabricated in the field, using a light platform of plywood as a working deck when applying the asphalt to the fibreglass. The fibreglass came in rolls three feet wide and four hundred feet long. Manufacturer's specifications describe this fibreglass mat as being 0.022 to 0.024 inches thick, weighing $1\frac{1}{2}$ pounds per 100 square feet, and having a minimum tensile strength per two inch width of 28 pounds parallel to the long fibres and 14 pounds transversely.

The plywood platform was covered with a layer of tarred building paper before the fibreglass strips were placed in a transverse direction to the centerline of the road. Each strip was overlapped one-half width in order to provide a double layer of fibreglass over the whole area. Air-blown roofing asphalt having a softening point of 140 degrees Fahrenheit was applied by hand at a rate of about 0.4 gallons

per square yard. The platform was moved as required by a winch line from a muskeg vehicle located ahead along the centerline of the road. (Figure 22).

Several observations could be made following the construction of this membrane, namely:

1. While this membrane could not be torn apart by hand if pulled in a transverse direction, there was a tendency for the seams to separate when the fill was placed. It was observed that a wave progressed in front of the advancing fill, placing longitudinal tension in the membrane, thereby opening several of the seams. This wave action can be noted in Figure 23.
2. Several punctures developed in the membrane when the weight of the fill caused settlement to take place, forcing roots into the membrane.
3. Sufficient width was not provided to turn the edges of the membrane up high enough to prevent water entering the fill from the sides.

In view of not having obtained a successful membrane with the original test layout, it was decided to try two further sections, modifying the construction procedures used in the light of the experience gained.

In these new sections the fibreglass was placed in both directions, in order to increase the tensile strength

of the membrane and to reduce the tendency for the seams to open. Catalytically blown asphalt was used to provide a more flexible membrane especially at low temperatures. The application rate for the first additional section was 0.58 gallons per square yard. This produced a relatively strong membrane close to one-quarter inch in thickness.

In the second additional section the quantity of asphalt was reduced to 0.32 gallons per square yard, with a two mil thickness of plastic placed on top of this asphalt fibreglass membrane.

General Observations

Details of these test sections are shown in Figure 24.

Initial savings in yardage of fill on the membrane sections, as noted immediately after construction, were more than offset by the additional yardage required to maintain the road in a condition to haul fill out to the failure section. In actual fact the membrane sections took larger quantities of fill than did the control section, but this was due to the softer terrain and hence greater settlements. The initial saving in yardage would be due to membranes delaying the movement of moisture into the fill, but since moisture did get in the effect was only temporary. Auger holes dug in each of the test sections approximately six weeks after

construction showed the water level to be the same in the fills as in the surrounding area.

It is considered that the second section of asphalt-impregnated fibreglass was as strong and durable a membrane as could be expected to be built under field conditions. In view of its failure to prevent the movement of water into the fill, it is felt that this method of improving the performance of light access roads is not feasible.

An analysis of the quantities of fill used on the various test sections and extrapolating to quantities per mile of twenty-four foot top width road yields the following interesting comparisons.

The control section built on AEI to BEI muskeg required a quantity of 23,000 cubic yards per mile. The first section of asphalt-impregnated fibreglass built on the softer FI muskeg required a rate of 30,000 cubic yards per mile. Both of these quantities are based on the total yardage of fill placed on each section in order to maintain the road suitable for hauling equipment. The net tonnage of fill hauled over these sections in a period of two weeks was approximately 5000 to 6500 tons, the latter amount being over the control section. This estimate does not include the weight of the trucks used for hauling.

The minimum quantity of 15,000 cubic yards per mile

would be based on the last section of the additional asphalt fibreglass membrane which was built on BEI muskeg. No hauling was done over this last section.

The total yardage for the whole project was 7036 cubic yards based on truck measurement, while by borrow pit survey the calculated figure was 5860 cubic yards. On this basis the above mentioned yardages, which were based on truck measurements, are approximately 20 per cent higher than they would be if yardages in original position were considered.

To get some indication of the cost of the various types of membranes, the approximate costs of the material only will be considered. The polyethylene plastic was $3\frac{1}{2}$ cents per square yard per mil of thickness, approximately 14 cents per square yard for the four mil thickness. The cost of the fibreglass, known commercially as Glassfalt-Type 200, was approximately 15 cents per square yard for a single thickness. Catalytically blown asphalt was approximately $60 per ton, which is equivalent to 30 cents per gallon. The combined material cost for the asphalt fibreglass membrane, considering a double layer of fibreglass and asphalt at 0.5 gallons per square yard, would be approximately 50 cents per square yard.

From this consideration it can be noted that the

overall economics of the matter of membranes and their possible advantages, decidedly limit their possible use in the future.

CHAPTER VI

SETTLEMENT OF FILLS ON MUSKEG

Observations of settlements on this project were directed towards obtaining information as to the magnitude and rate of settlement during and shortly after construction. With continued hauling over the membrane sections, construction equipment damaged the settlement gauge pipes which were subsequently buried beneath additional fill. The same could be said for many of the settlement gauges on the fill to failure section. In view of this, most of the information obtained was limited to relatively short term periods of observation.

On the first light fill control and the membrane sections, the amount of settlement was greater on the FI than on the BEI muskeg. It was found that the fill had sufficient strength to carry the construction equipment when the top of the grade was about two foot above the water surface. When fill was spread to this two foot height there was an almost immediate settlement of one foot on the BEI and about two feet on the FI muskeg. The actual amount of fill placed was then three to four feet. Settlements after one day were two and four feet respectively. Further observations for a period of up to two weeks and a

test auger hold through the fill sixty days later indicated that settlement was progressing at a rate proportional to the log of time. This rate on the FI muskeg was 0.11 feet per foot of depth per cycle of time in days. The load was approximately 0.24 tons per square foot. With the observed settlement of 6.0 feet this was 55 per cent of the 10.5 foot total depth of muskeg. (Figure 25).

Piezometers set at the eight to ten foot depths indicated excess pore pressures of four to five feet of water immediately after the fills were built. Observations of pore pressures in piezometer No. 1 showed that the excess pressure of 4.2 feet after placing an additional one foot of fill dissipated in approximately four days. (Figure 26).

Laboratory consolidation tests were run on two samples of the FI muskeg, one from the one foot depth and the other from the eleven foot depth. Figures 27 and 28 show the results of these tests. A plot of dial reading versus log time for various loads up to 2.0 tons per square foot does not give the typical shape of curve for inorganic soils, exhibiting primary and secondary branches, but does indicate a somewhat straight line relationship. The shape of these time curves are very similar to those reported by Thompson and Palmer.[10] In order to determine whether "primary" or "secondary" consolidation was taking place in

the laboratory test, data obtained from field observations of pore pressures was used. Considering that primary consolidation was over when the excess pore pressures were dissipated, and that the time for consolidation varies as the square of the length of drainage path, the theoretical time required to complete the primary branch in the laboratory test was then computed. Using the observed time of four days and a drainage path of eight feet in field conditions, the length of time for the 1.4 inch high specimen drained both ways would be two seconds. With a six second reading being the first one taken this would indicate that the observed laboratory readings were taken on the secondary time branch.

Whether or not this concept of secondary consolidation which continues at a rate proportional to the log of time independently of drainage conditions can be applied to settlement of fills on muskeg requires further investigation. (11), (12).

Additional observations on the remaining settlement gauges may give further information in this regard.

Field observations of settlement gauges installed at a tank farm in the Pembina oil field in September 1956[3] indicate that the settlement is progressing at this rate proportional to the log time although being much lower. At

this particular location the muskeg was 5.8 feet deep with the surface cover described as ABI. Under the load of 0.15 tons per square foot the initial consolidation was 1.17 feet or 21 per cent of the total depth. Total consolidation up to September 1958 was 1.35 feet, giving the rate of secondary consolidation of 0.017 feet per foot of depth per cycle of time in days. (Figure 29).

A general observation that could be made following the construction of these test fills is that large settlements take place during and shortly after the fill is placed. The amount of settlement could vary from 10 to 50 per cent of the depth of muskeg depending on the weight of fill placed and type of muskeg. The surface cover formulae can be very helpful in predicting the relative amount of settlement to be expected.

CHAPTER VII

SUMMARY OF VANE SHEAR STRENGTH DATA

The vane shear strengths measured were somewhat lower than those obtained by previous investigators Thomson[2] and Wyld.[3] This is due to the different type of muskeg tested.

The majority of vane shear tests were run on FI type muskeg approximately twelve feet in depth. The average undisturbed shear strength increased from 125 pounds per square foot at the three foot depth, to 200 pounds per square foot at the eleven foot depth.

Several tests on BEI type muskeg gave corresponding values of 225 pounds per square foot at the three foot depth, decreased to 175 at the eight foot depth, and then increased to 225 again at the eleven foot depth.

A summary of results obtained by Thomson and Wyld together with this study, is shown in tabular form on the following page. Several pertinent comparisons are noted:

1. Wyld noted that for DFI muskeg, there was a general increase in strength with depth, with a characteristic decrease in strength just below mid-depth with the minimum strength recorded at this point. A similar trend was observed to occur on BEI muskeg in this

current study. This seems to corroborate the observation made earlier that the larger surface growth has a decided effect on the moisture content and hence the shearing strength of the underlying organic material.

2. FI type muskeg has the lowest strength and highest moisture content of any tested to date.

Radforth Class	Depth to Bottom ft.	Typical Strengths $\#/ft^2$	Range of Moisture Contents %	Area Investigated
BEI	12 17.5	520 210 to 1090	- -	Alaska (i) Highway
DFI	10.5	270 to 390	800 to 500	Pembina (ii)
BDI	9.5	150 to 575	900 to 700	"
ADI	6.5	275 to 475	-	"
FI	12.0	125 to 200	1300 to 900	Alsike (iii)
BEI	12.0	175 to 225	1000 to 600	"

(i) Thomson
(ii) Wyld
(iii) Anderson

Summary of Shear Strengths on Muskeg
University of Alberta Investigations

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The moisture content, as an index property, is the best indicator of the engineering properties and behaviour of muskeg. The moisture content is closely related to the shearing strength and compressibility of muskeg, and therefore can be used to anticipate and take into account in design, problems pertaining to road construction on muskeg.

The shearing strength of muskeg as measured by the particular vane shear test apparatus used, can be used in analyzing the stability of fills constructed on FI type muskeg.

The membranes of polyethylene plastic and asphalt-impregnated fibreglass placed directly on the surface of the muskeg were not successful, in that they failed to prevent the movement of moisture into the constructed earth fills.

Large settlements of fills constructed on muskeg must be expected and taken account of in design. Standard consolidation test procedures developed for inorganic soils cannot be applied directly to predict the amount of

settlement to be expected.

The Radforth Classification System can be very useful in predicting the engineering properties of various types of muskegs, in that the surface cover influences the moisture content of the underlying material. Having a knowledge of the surface cover by the various formulae the relative values of shearing strength and compressibility can be estimated.

Recommendations

The problem of heavier fills constructed on muskeg should be further investigated. Co-operative research programs with Highway Departments or other interested parties engaged in road construction on muskeg should be arranged in order to derive useful information from routine construction projects. In particular, vane shear strength tests should be taken on muskeg areas where the design height of fill is six feet or over. This information would be helpful in planning construction procedures, and very informative in the event of failure.

Further information on the consolidation characteristics of muskeg could also be obtained by the installation of settlement gauges and piezometers in some selected fills on various types and depths of muskeg areas. In conjunction with field programs, investigations should be directed toward developing laboratory test techniques to estimate the amount and rate of settlement of fills on muskeg.

BIBLIOGRAPHY

1. Radforth, N.W., and MacFarlane, I.C. "Correlation of Palaeobotanical and Engineering Studies of Muskeg (Peat) in Canada, "Reprint from 4th International Conference on Soil Mechanics and Foundation Engineering, London, 1957. Research Paper No. 35, Division of Building Research, Ottawa.

2. Thomson, Captain S. "An Investigation of the Engineering Properties of Muskegs," Unpublished Master's thesis, University of Alberta School of Graduate Studies, 1955.

3. Wyld, Captain R.C. "A Further Investigation Into the Engineering Properties of Muskeg," Unpublished thesis, University of Alberta School of Graduate Studies, 1956.

4. ASTM Special Technical Publication No. 193. "Symposium on Vane Shear Testing of Soils."

5. Mecham, J.G. "A Vane Borer for Testing the Stability and Strength of Soil Subgrades and Soil Foundations," University of Idaho, Bulletin No. 11, 1956.

6. Casagrande, A. "Non-metallic Piezometer for Measuring Pore Pressures in Clay," Journal of The Boston Society of Civil Engineers, April 1949, Reprint, Contributions to Soil Mechanics, 1941-53, pl. 98. Also revisions prepared in February 1958.

7. Waters, J.M. and Bartlett, D.L. "A Direct Method for the Location of Slip Planes," Reprint, Civil Engineering and Public Works Review, 8 Buckingham Street, London, W.C.2. September 1956.

8. Terzaghi, K. and Peck, R.B. "Soil Mechanics in Engineering Practice," John Wiley and Sons, New York.

9. Jurgenson, Leo. "Application of Theories of Elasticity and Plasticity to Foundation Problems," Journal of the Boston Society of Civil Engineers, July 1934, Reprint, Contributions to Soil Mechanics 1925-40, p.170.

10. Thompson, J.B. and Palmer, L.A. "Report of Consolidation Tests with Peat," ASTM Special Technical Publication No. 126.

11. Rutledge, P.C. and Johnson, S.J. "Review of Uses of Vertical Sand Drains," Highway Research Board Bulletin No. 173.

12. Hanhrahan, E.T. "An Investigation of Some Physical Properties of Peat," Geotechnique, September 1954.

APPENDIX I

PROPERTIES DESIGNATING NINE PURE COVERAGE CLASSES

(from Radforth Muskeg Classification)

Coverage Type (Class)	Woodiness vs. Non-woodiness	Stature (approx. height)	Texture (where req'd)	Growth Habit	Example
A	woody	15 ft. or over	------	tree form	Spruce Larch
B	woody	5 to 15 ft.	------	young or dwarfed tree or bush	Spruce Larch Willow Birch
C	non-woody	2 to 5 ft.	------	tall grass-like	Grasses
D	woody	2 to 5 ft.	------	tall shrub or very dwarfed tree	Willow Birch Labrador tea
E	woody	up to 2 ft.	------	low shrub	Blueberry Laurel
F	non-woody	up to 2 ft.	------	mats, clumps or patches, sometimes touching	Sedges Grasses
G	non-woody	up to 2 ft.	------	singly or loose association	Orchid Pitcher plant
H	non-woody	up to 4 in.	leathery to crisp	mostly continuous mats	Lichens
I	non-woody	up to 4 in.	soft or velvety	often continuous mats, sometimes in hummocks	Mosses

APPENDIX II

THEORY OF VANE SHEAR TEST APPARATUS

The resisting moment to the torque applied is developed by the shearing strength of the soil acting on the revolved area of the vane. The following assumptions are made:

1) The surface of rupture is a circular cylinder surrounding the vane with a diameter D and height H equal to the dimensions of the vane.

2) The stress distribution at the maximum torsional moment is uniform over the surface of the cylinder and on the end areas varies uniformly from this same value to zero at the shaft centerline.

This resisting moment can be expressed by the equation

$$T = S \times A \times M \text{ ....equation (1)}$$

where T = torque applied

S = shearing strength of soil

A = revolved area of vane

M = moment arm of the centroid of the element of the shear surface.

When it is necessary to advance an auger hole to the desired depth of test less one half the length of the vane and then to penetrate the vane to its full length, the area on which the shear stress acts is the cylindrical wall area plus one end area.

Expressing equation (1) in terms of the diameter D and the length of vane H,

$$T = S\left[\left(\pi D \times H \times \frac{D}{2}\right) + \left(\frac{\pi D^2}{4} \times \frac{2}{3} \times \frac{D}{2}\right)\right]$$

$$= \frac{\pi D^2}{2}\left(H + \frac{D}{6}\right) \times S$$

or $\quad S = K{\times}T \quad$ where $K = \dfrac{1}{\dfrac{\pi D^2}{2}\left(H + \dfrac{D}{6}\right)}$

With the vane dimensions of D = 4.5 inches, H = 10.1 inches.

$$K = \frac{1}{0.20} = 5.0 \quad (\text{units of } 1/ft^3)$$

Therefore S = 5.0 T . . . . equation (2)

With the vane advanced without an auger hole being drilled, the area on which the shear stress acts is the cylindrical wall area plus two end areas. Neglecting the loss of the area of the shaft,

$$T = S\left[\frac{\pi D^2}{2}\left(H + \frac{D}{3}\right)\right]$$

$$S = K'\,T \quad \text{where } K' = \frac{1}{0.213} = 4.7$$

$$S = 4.7\,T \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad \text{equation (3)}$$

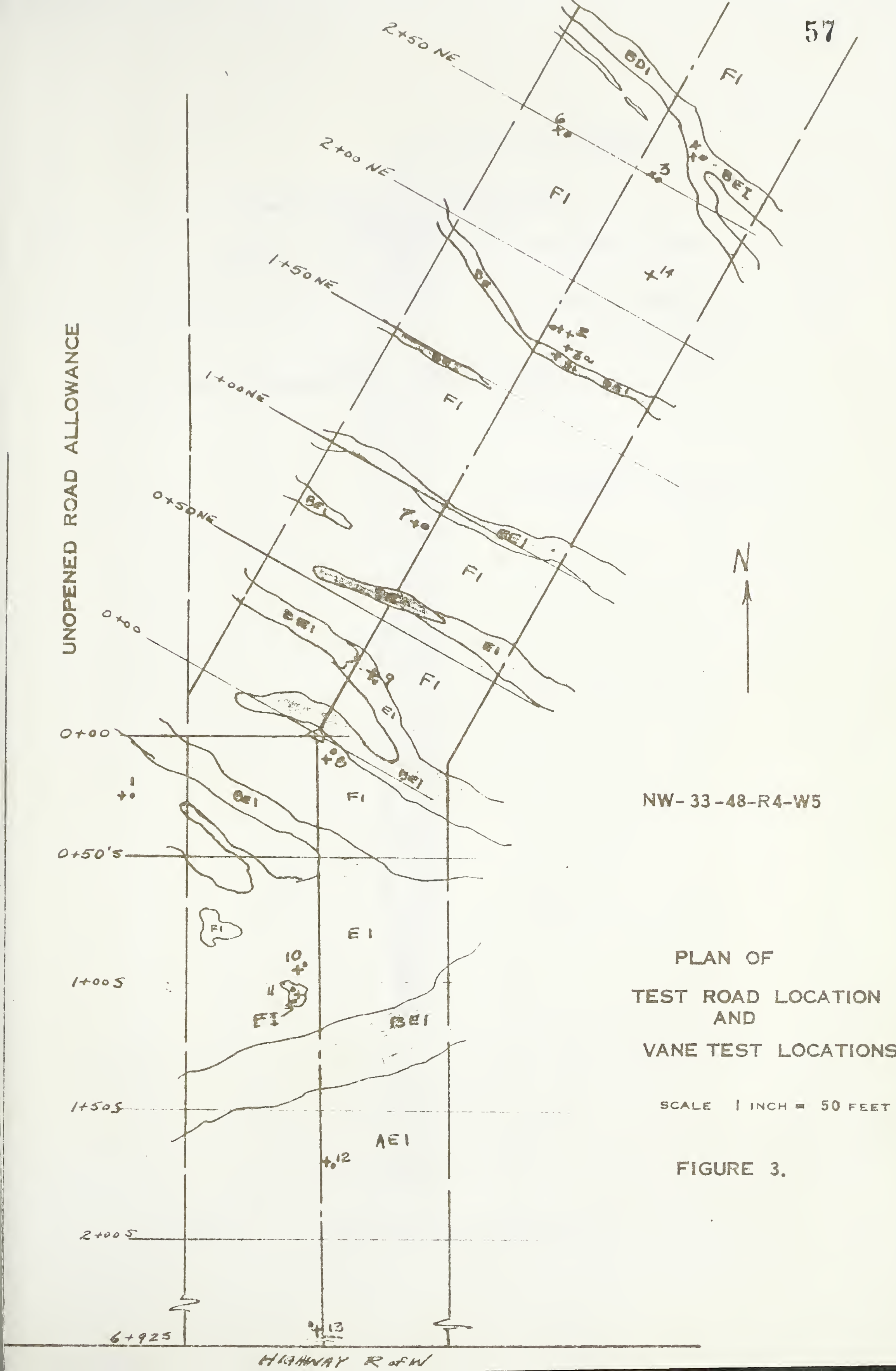
These relationships are expressed graphically in Figure 6.

FIGURE 1

Approximate Scale 4" = 1 Mile

Reproduced by courtesy of the Technical Division of the Department of Lands and Forests, EDMONTON, Alberta.

FIGURE 2



NW-33-48-R4-W5

PLAN OF

TEST ROAD LOCATION
AND
VANE TEST LOCATIONS

SCALE 1 INCH = 50 FEET

FIGURE 3.

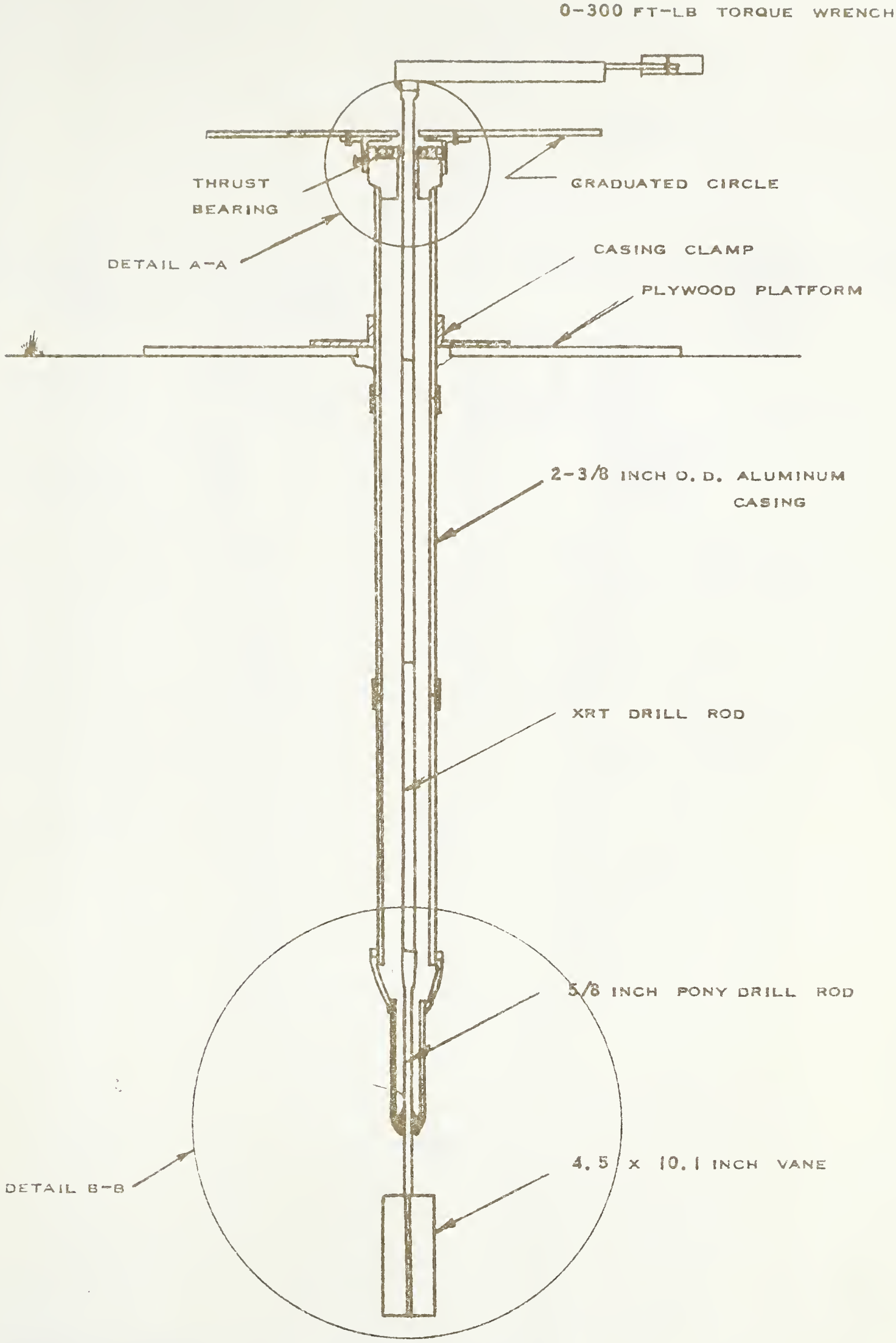


FIGURE 4 VANE SHEAR APPARATUS

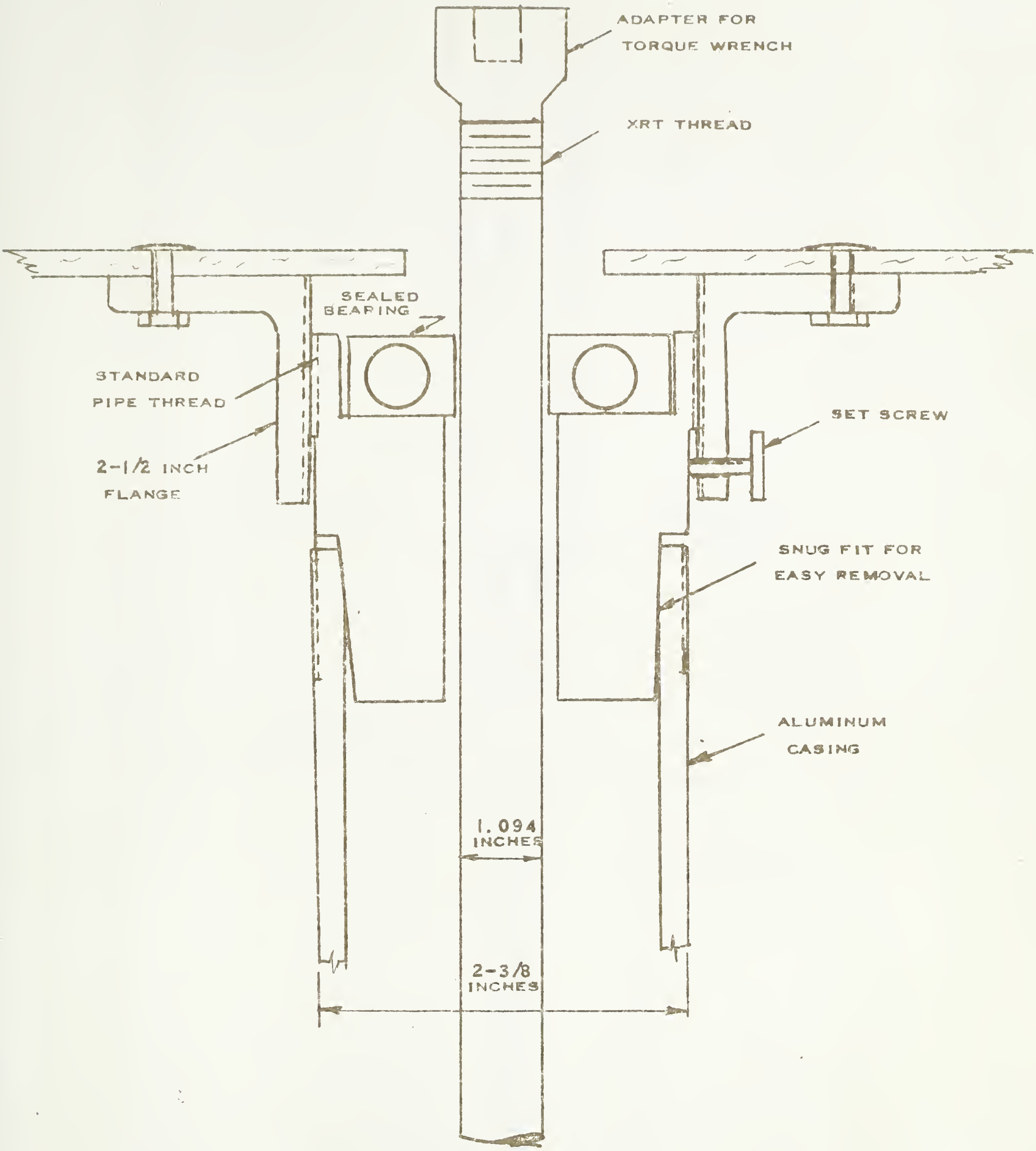


FIGURE 4- DETAIL A–A   THRUST BEARING

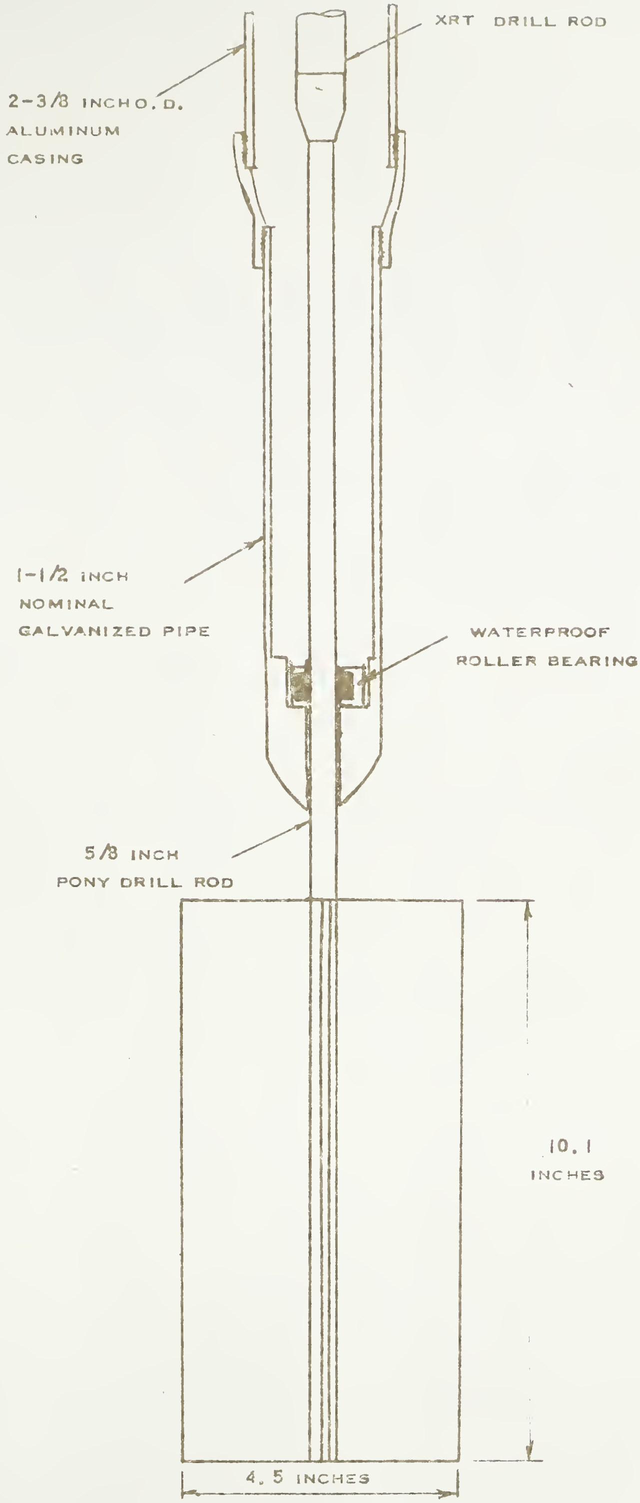


FIGURE 4- DETAIL B-B VANE ASSEMBLY

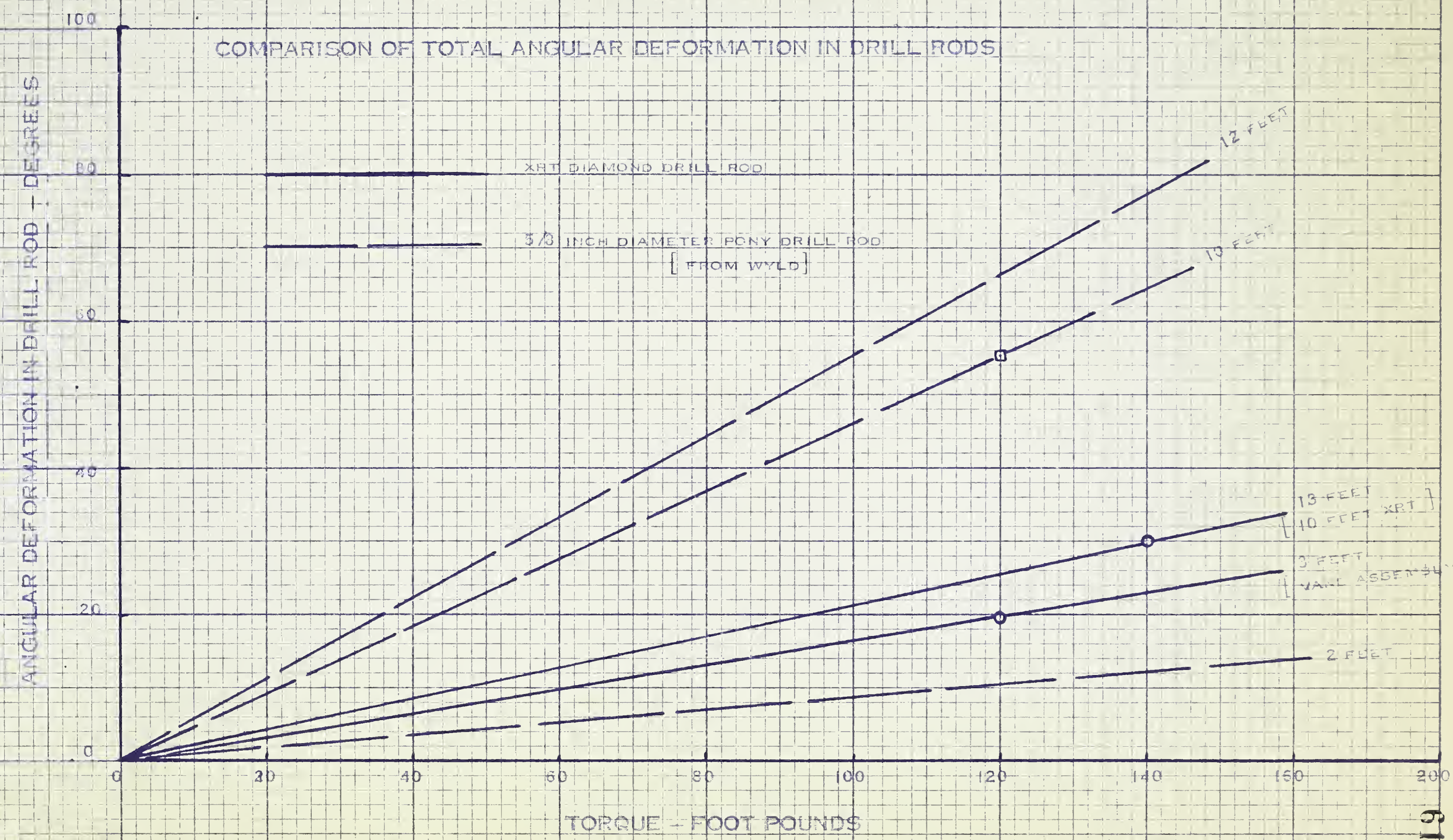

COMPARISON OF TOTAL ANGULAR DEFORMATION IN DRILL RODS
XRT DIAMOND DRILL ROD
5/8 INCH DIAMETER PONY DRILL ROD
[FROM WYLD]
12 FEET
10 FEET
13 FEET
[10 FEET XRT]
3 FEET
[VANE ASSEMBLY]
2 FEET
ANGULAR DEFORMATION IN DRILL ROD - DEGREES
0
20
40
60
80
100
0
20
40
60
80
100
120
140
160
200
TORQUE - FOOT POUNDS

FIGURE 5.

TORQUE CHART FOR VANE SHEAR TEST APPARATUS

RECTANGULAR VANE

LENGTH = 10.1 INCHES
DIAMETER = 4.5 INCHES

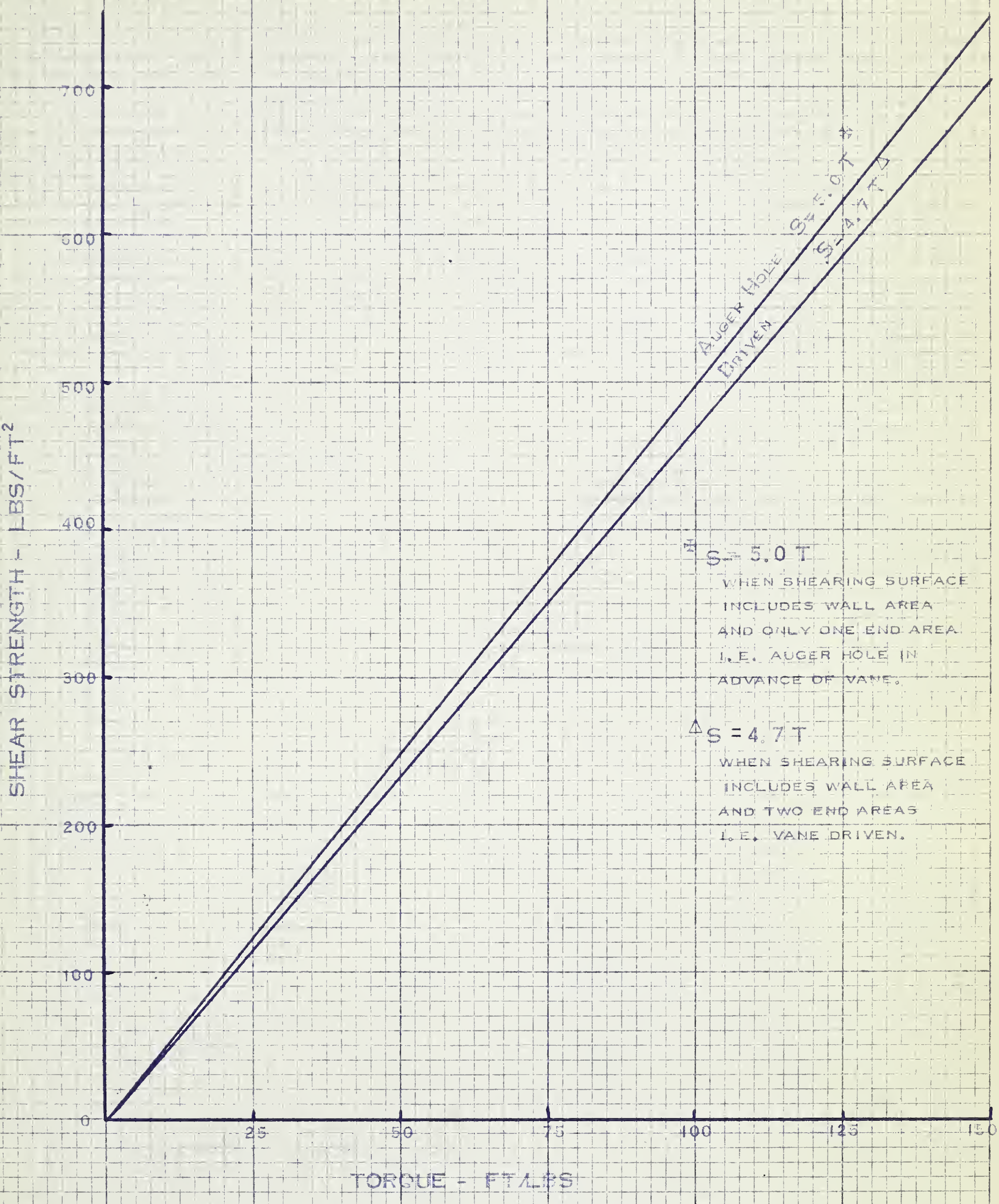


FIGURE 6

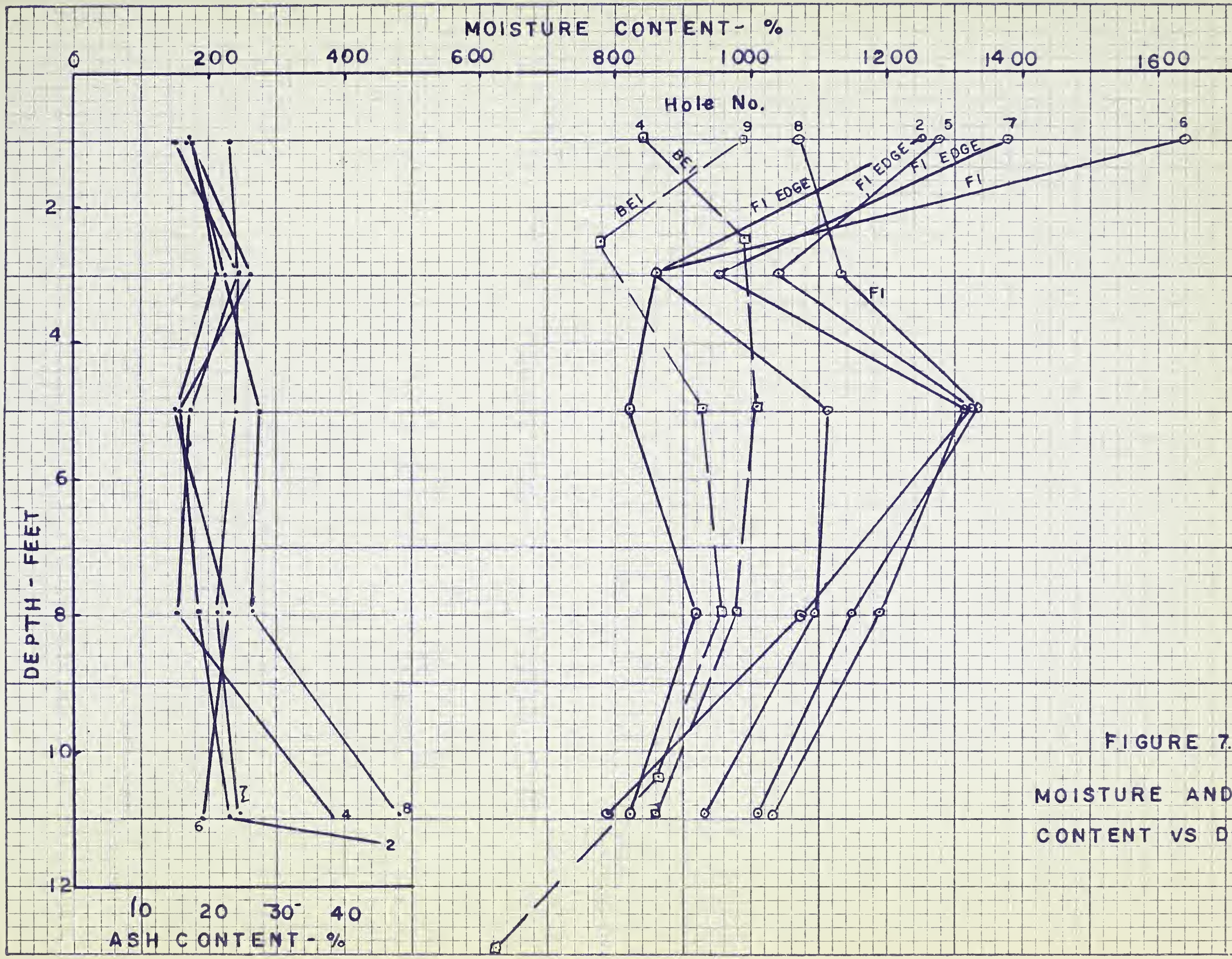


FIGURE 7.

MOISTURE AND ASH CONTENT VS DEPTH

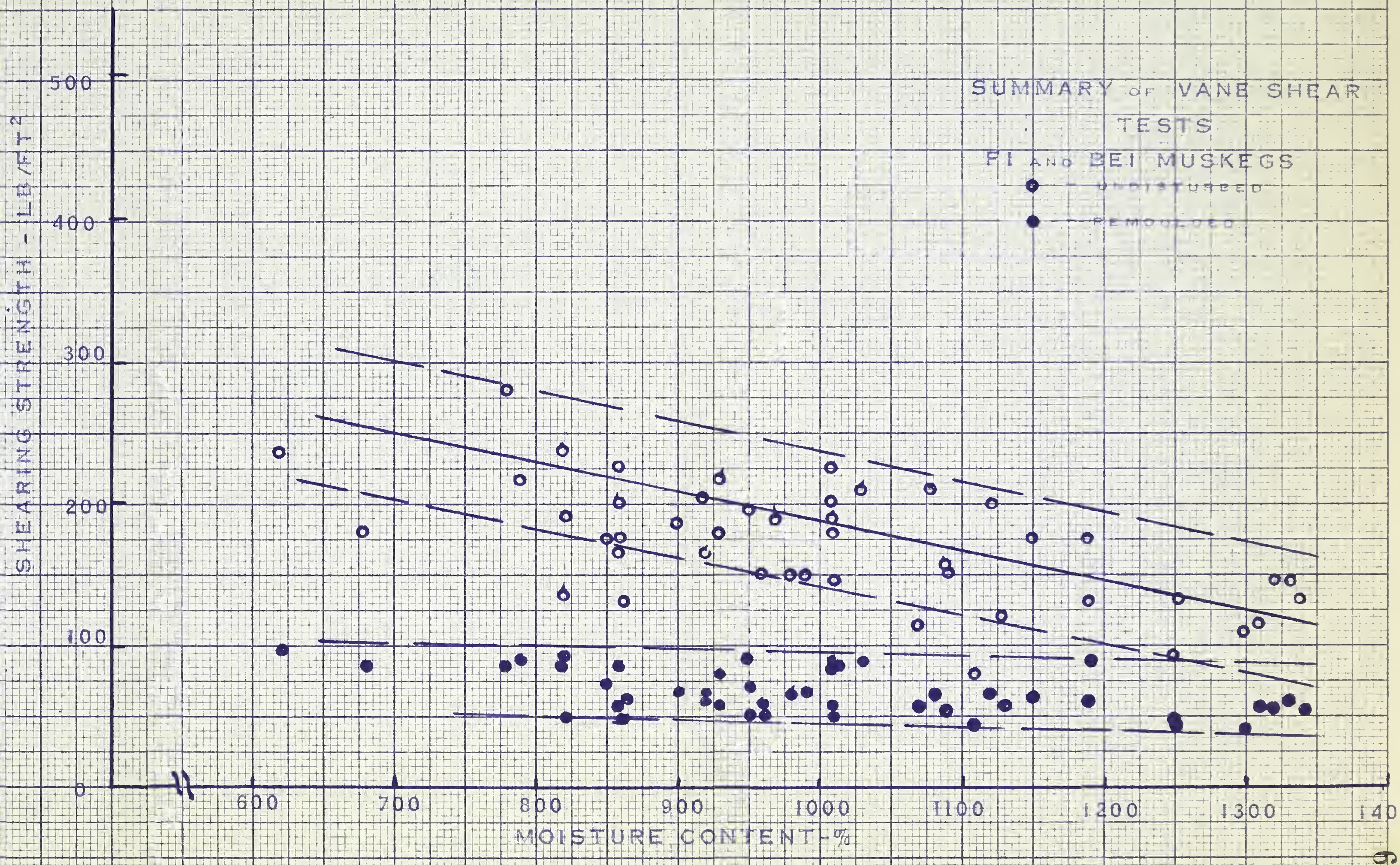

SUMMARY OF VANE SHEAR TESTS
FI AND BEI MUSKEGS
UNDISTURBED
REMOULDED
SHEARING STRENGTH - LB/FT^2
500
400
300
200
100
0
600
700
800
900
1000
1100
1200
1300
1400
MOISTURE CONTENT - %

FIGURE 8

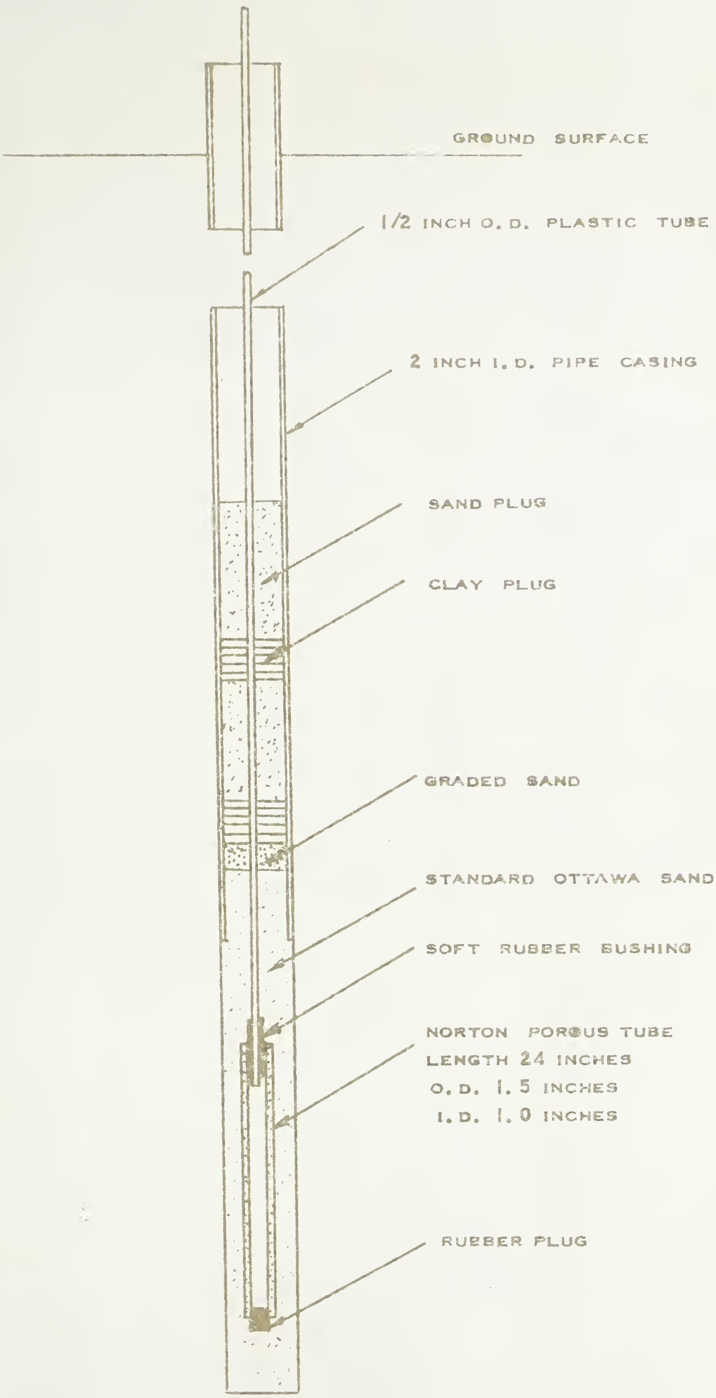


FIGURE 9. SCHEMATIC DIAGRAM OF PIEZOMETER INSTALLATION

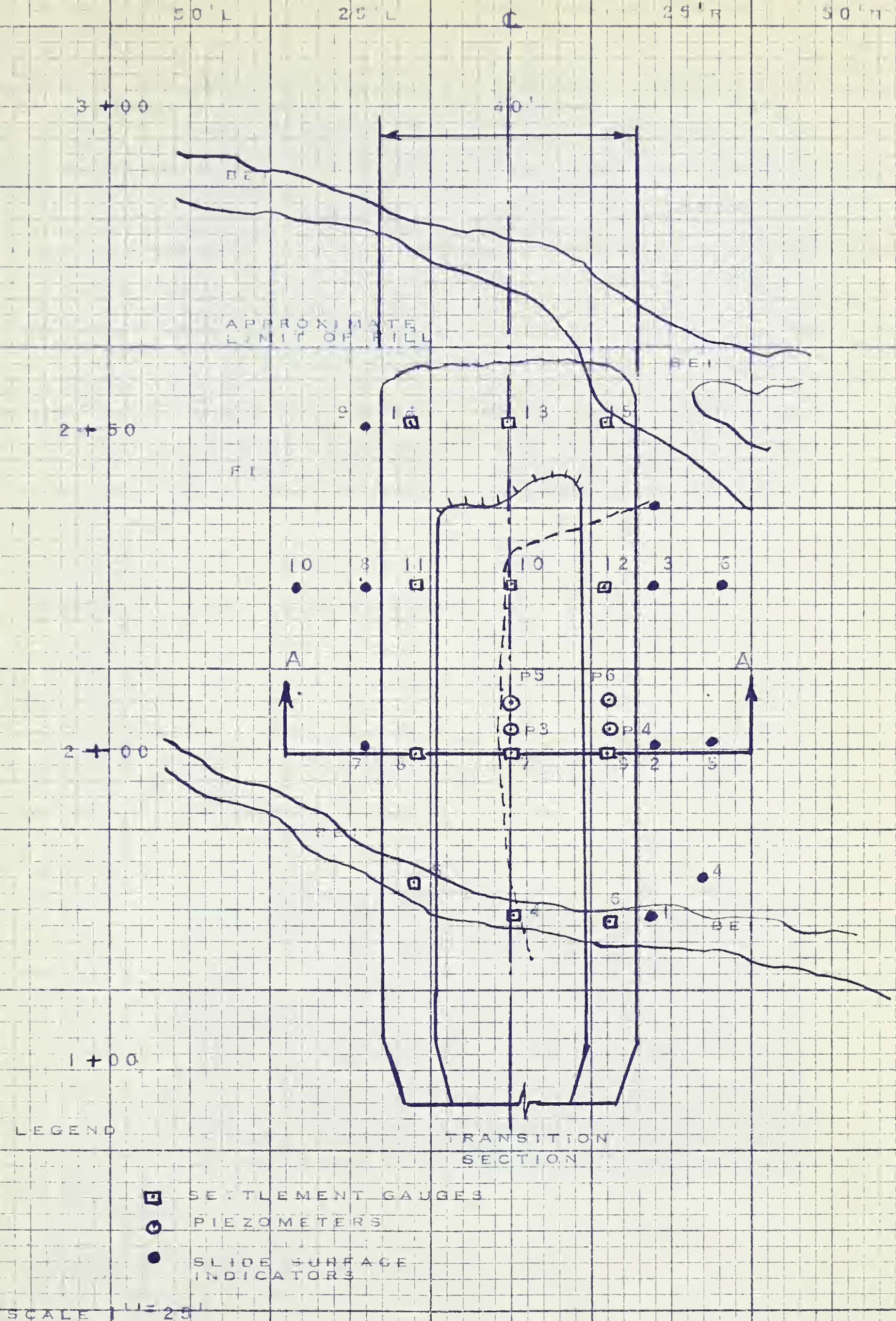


FIGURE 10. PLAN OF FAILURE SECTION

FIGURE 11. INSTRUMENTATION ON FAILURE SECTION

FIGURE 12. START OF FAILURE SECTION

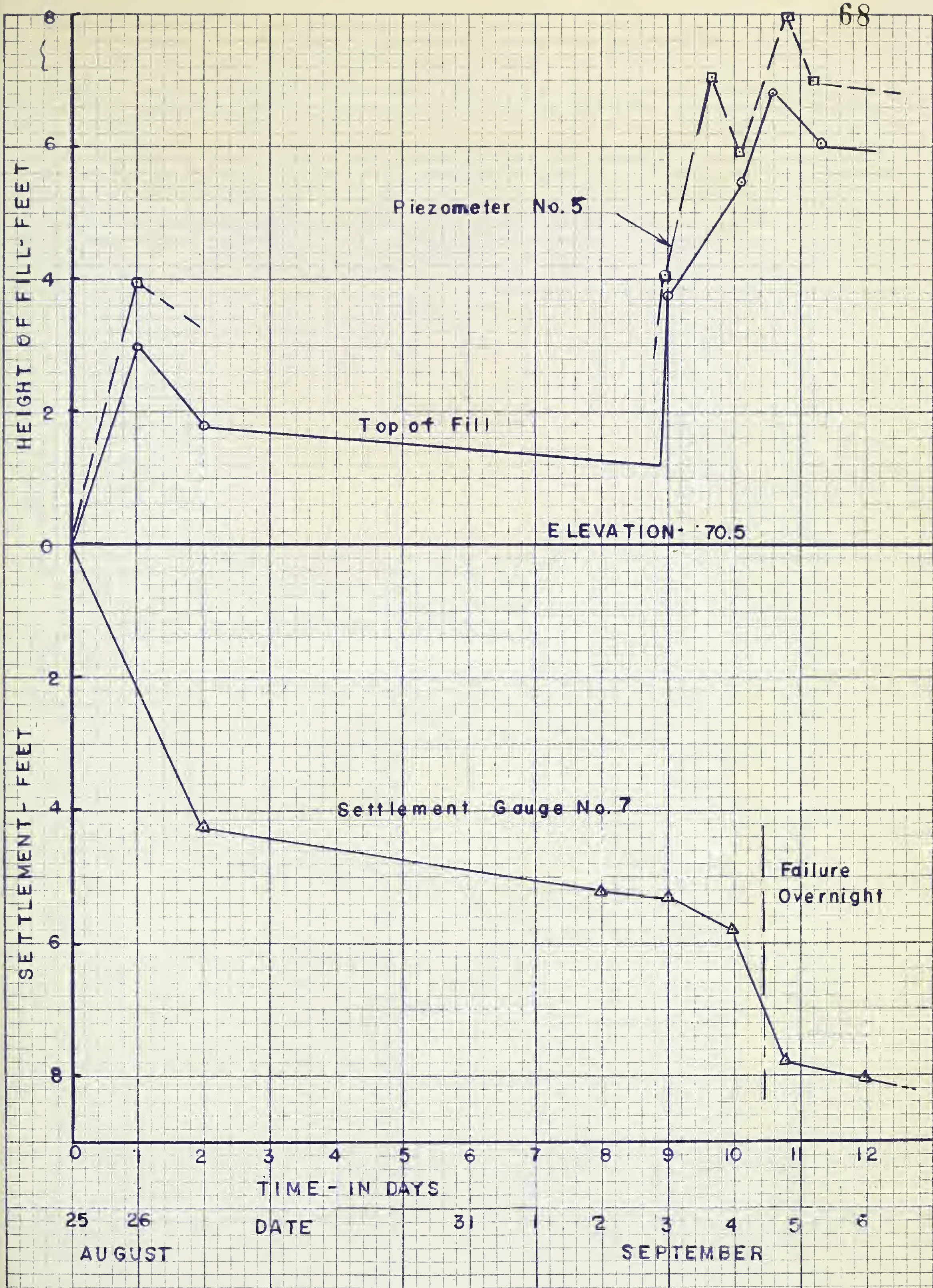
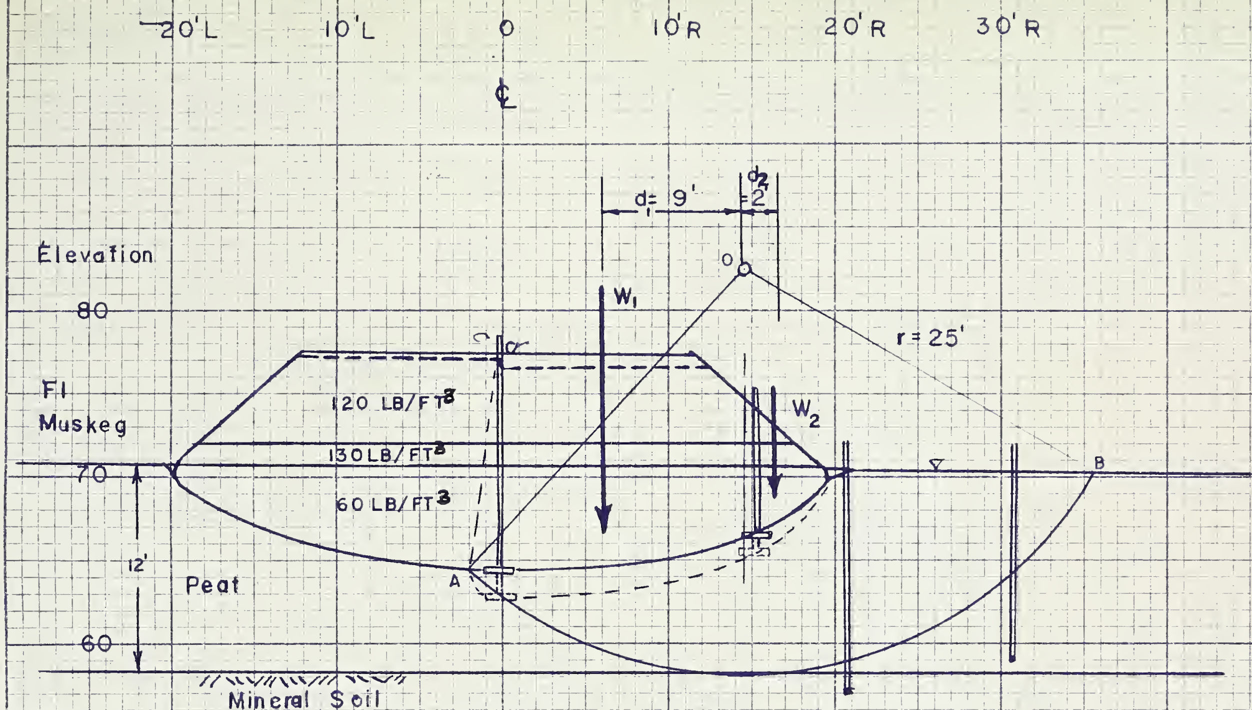


FIGURE 13. LOADING FILL TO FAILURE



Section A-A Sta. 2+00

$$S = \frac{W_1 d_1 - W_2 d_2}{l \cdot r}$$

$W_1 = 18000$ LB.

$W_2 = 3700$ LB

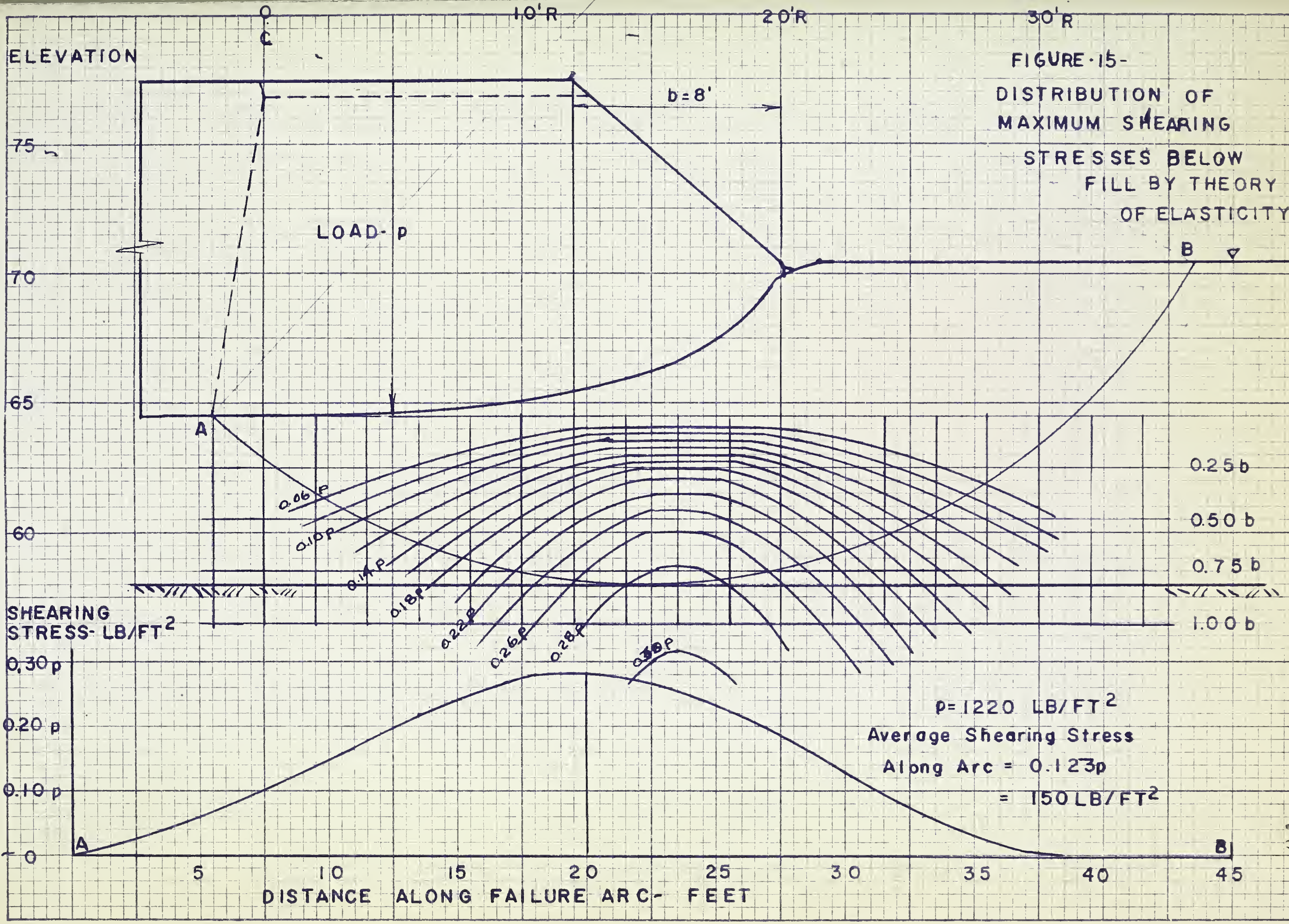
$l = 45'$ $r = 25'$

Average Shearing Resistance

Along Arc AB, $S = 160\ \text{LB/FT}^2$

FIGURE 14.

CIRCULAR ARC ANALYSIS


FIGURE·15-
DISTRIBUTION OF MAXIMUM SHEARING STRESSES BELOW FILL BY THEORY OF ELASTICITY
ELEVATION
75
70
65
60
0
10'R
20'R
30'R
C
b=8'
LOAD- p
A
B
0.25 b
0.50 b
0.75 b
1.00 b
0.06 p
0.10 p
0.14 p
0.18 p
0.22 p
0.26 p
0.28 p
SHEARING STRESS- LB/FT^2
0.30 p
0.20 p
0.10 p
0
p=1220 LB/FT^2
Average Shearing Stress
Along Arc = 0.123p
= 150 LB/FT^2
5
10
15
20
25
30
35
40
45
DISTANCE ALONG FAILURE ARC- FEET

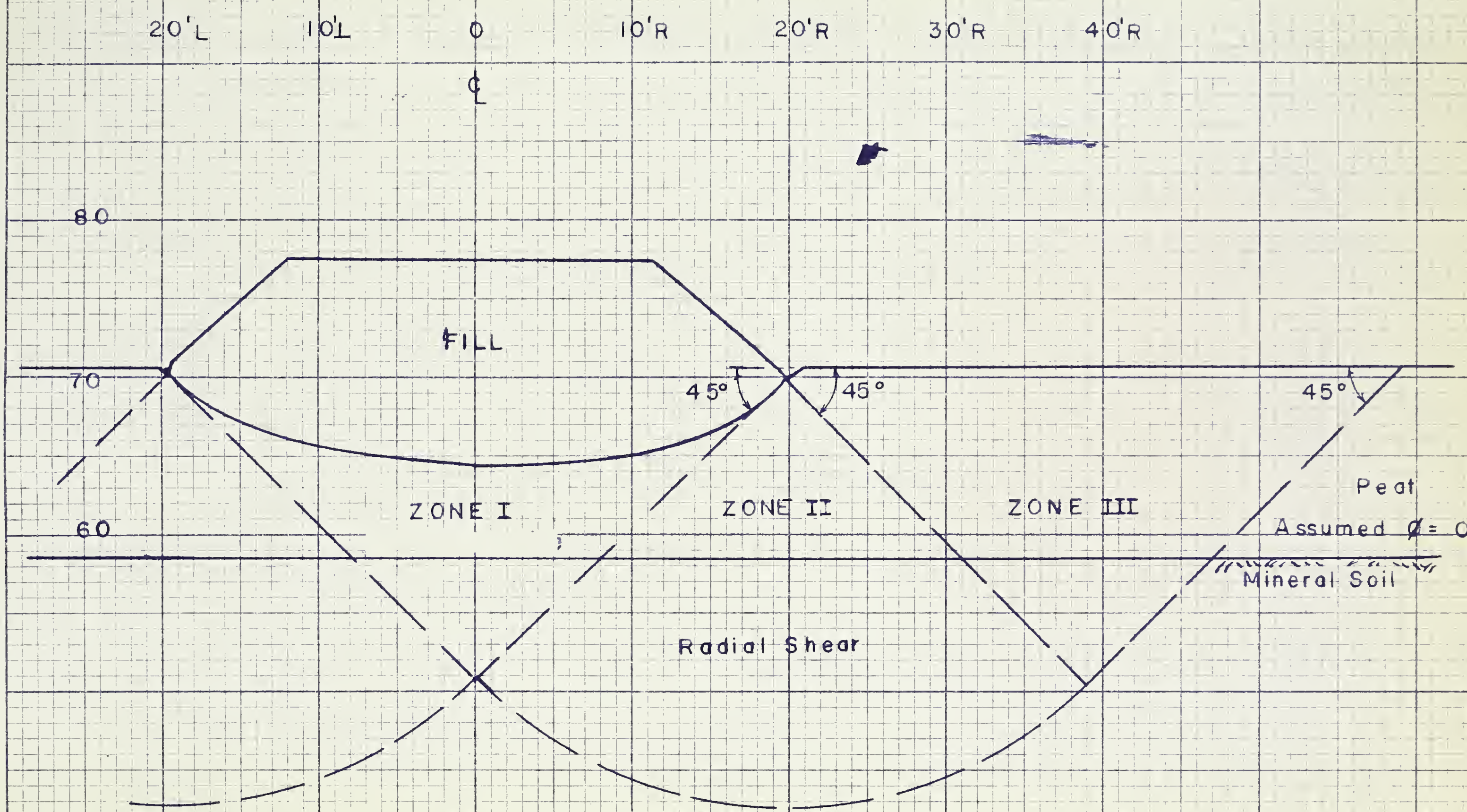
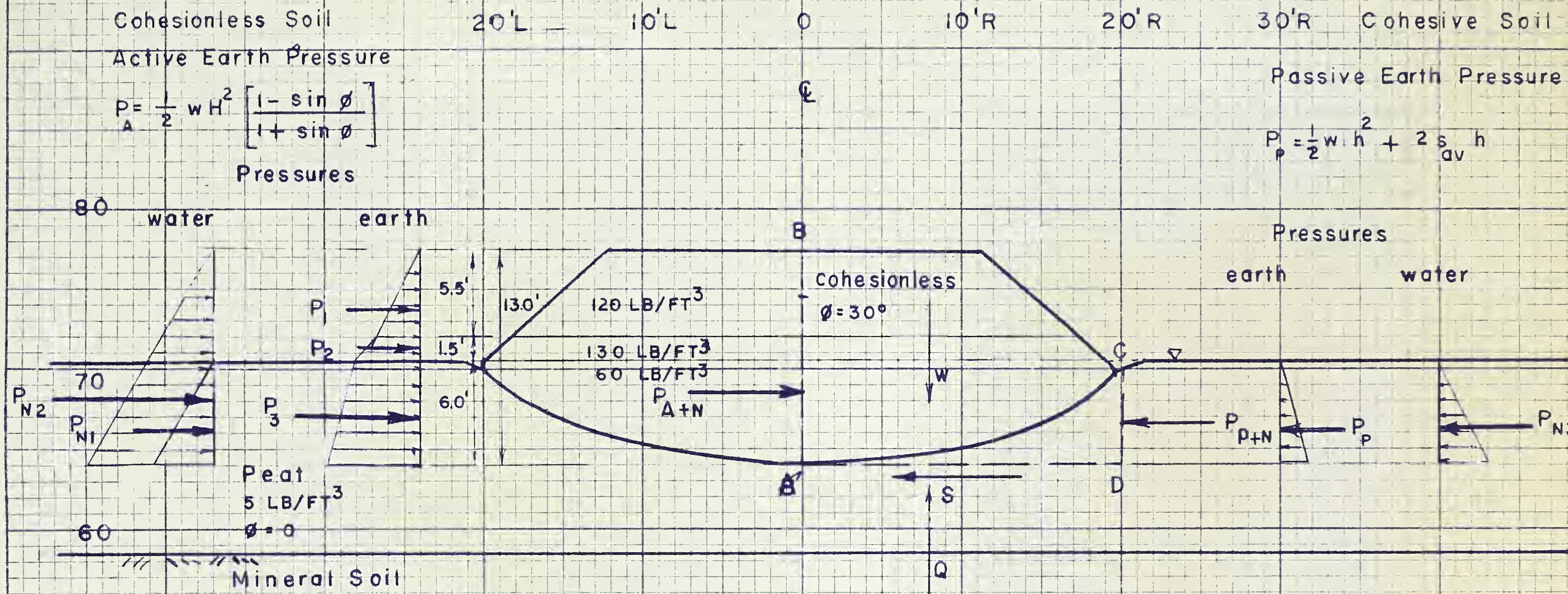


FIGURE 16. BEARING CAPACITY BY PLASTIC EQUILIBRIUM THEORY



Case I - No excess water pressure

Forces in pounds

$P_1 = 605$
$P_2 = 380$
$P_3 = 2070$
$P_{N1} = 1120$

$P_p = 90 + 12\, s_{av}$
$S = 20\, s_{av}$
$P_{N3} = 1120$

Required $s_{av} = 95$ LB/FT^2

Case II. 8.0 Feet excess water pressure on lower 10.0' surface AB.

$P_A = 3055$
$P_{N2} = 5630$

Required $s_{av} = 235$ LB/FT^2

Case III. Excess water pressure as in II
~~Earth pressure drop 3.0'~~

$P_A = 180$
$P_{N2} = 5630$

Required $s_{av} = 140$ LB/FT^2

FIGURE 17. SLIDING BLOCK ANALYSIS

FIGURE 18. CONTROL SECTION

FIGURE 19. WELDING PLASTIC MEMBRANE

FIGURE 20. PLACING FILL ON PLASTIC MEMBRANE

FIGURE 21. TEARING OF JOINTS ON PLASTIC MEMBRANE

FIGURE 22. ASPHALT FIBREGLASS MEMBRANE

FIGURE 23. PLACING FILL ON ASPHALT FIBREGLASS MEMBRANE

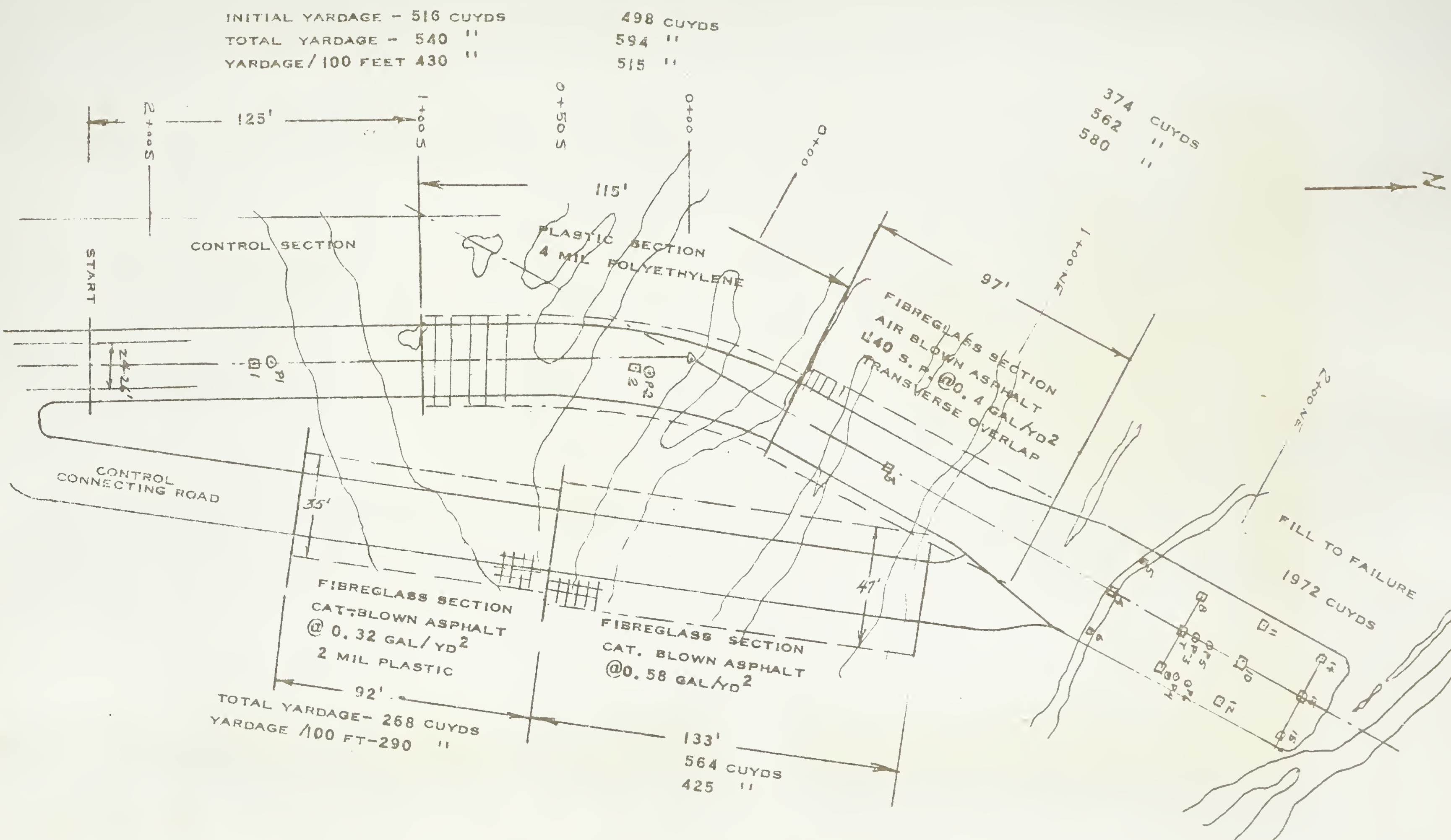


FIGURE 24. DETAILS OF TEST SECTIONS

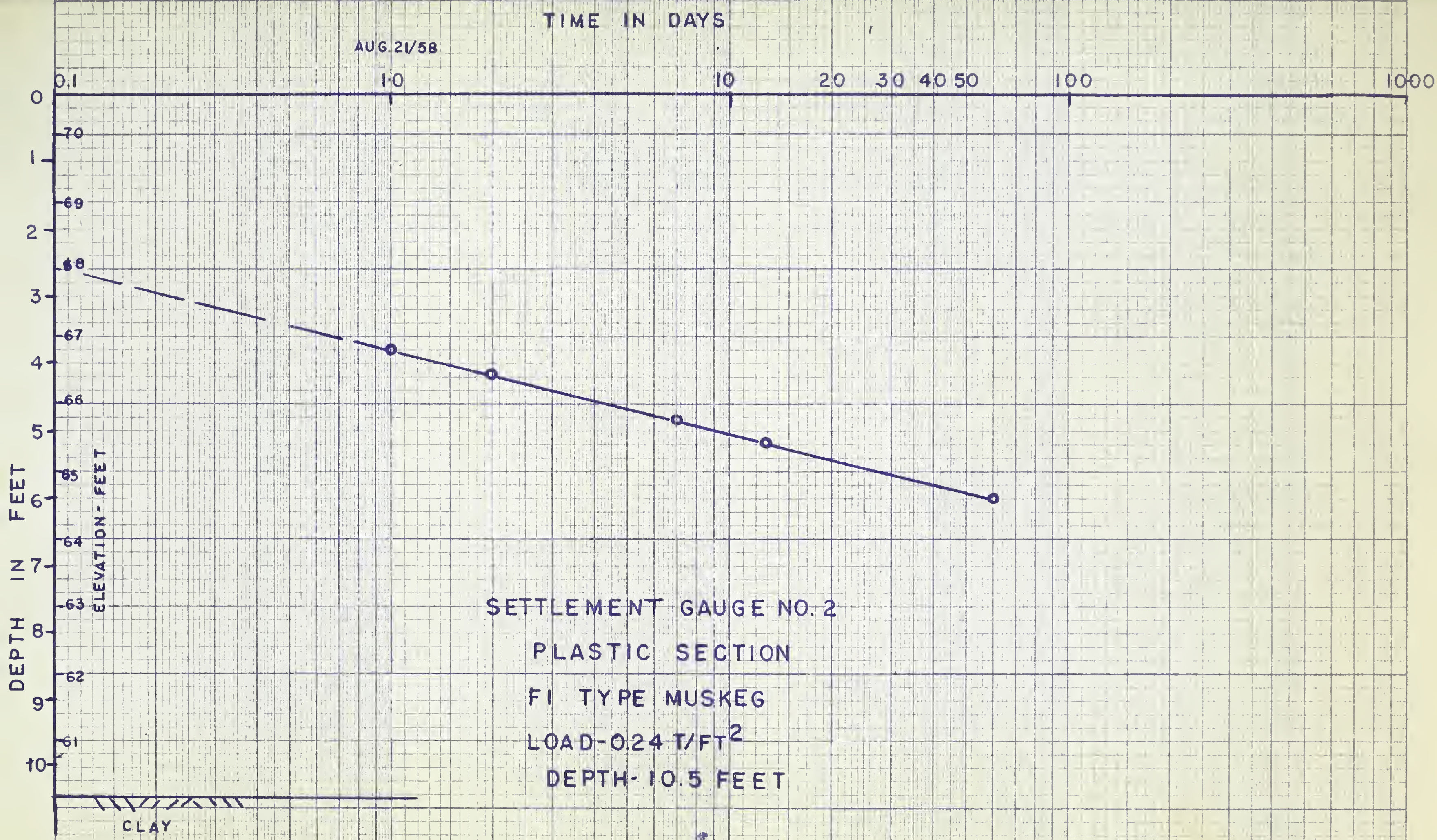
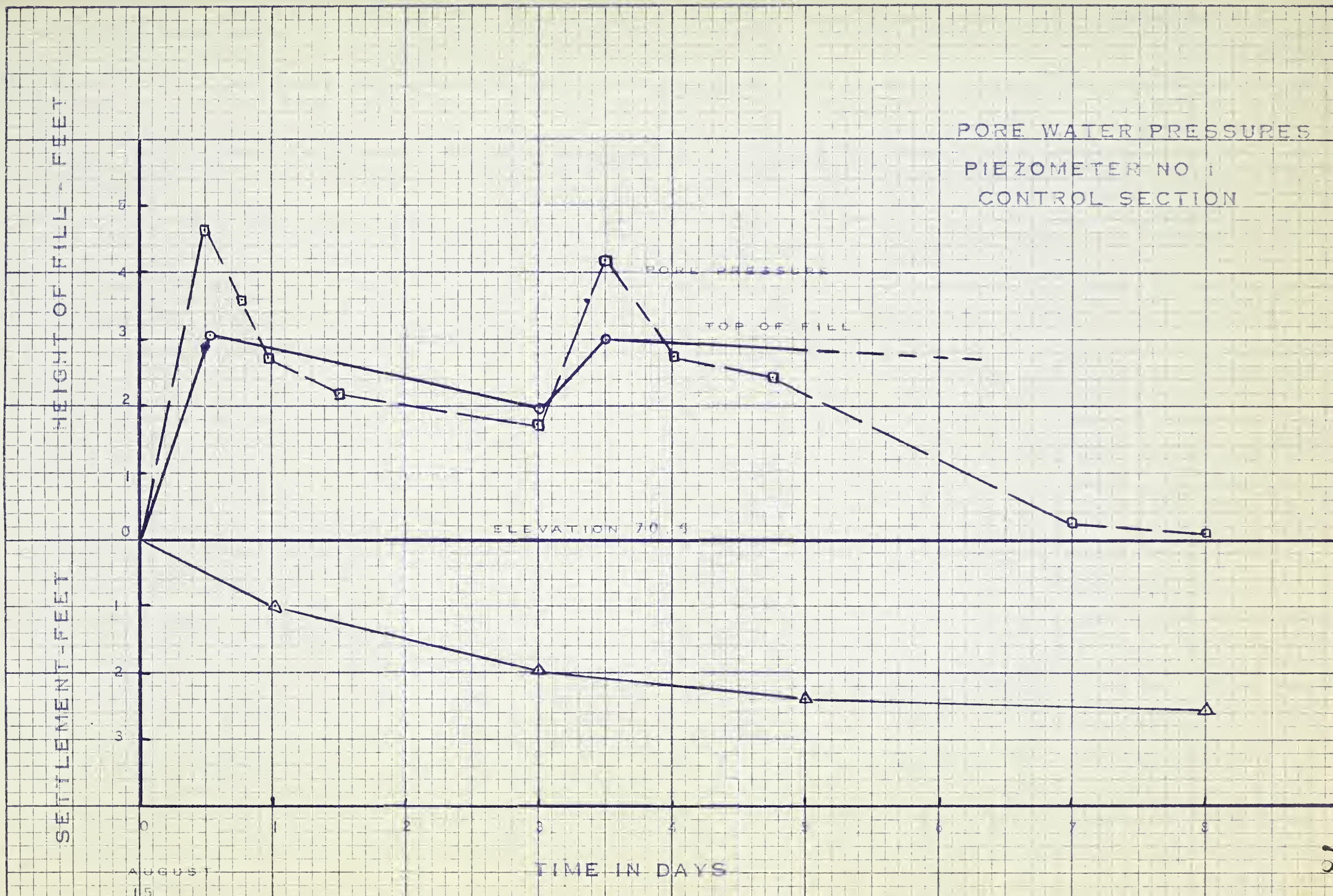

TIME IN DAYS
AUG. 21/58
0.1
1.0
10
20
30
40
50
100
1000
DEPTH IN FEET
0
1
2
3
4
5
6
7
8
9
10
ELEVATION - FEET
70
69
68
67
66
65
64
63
62
61
CLAY
SETTLEMENT GAUGE NO. 2
PLASTIC SECTION
FI TYPE MUSKEG
LOAD-0.24 T/FT²
DEPTH-10.5 FEET

FIGURE. 25



PORE WATER PRESSURES
PIEZOMETER NO 1
CONTROL SECTION
PORE PRESSURE
TOP OF FILL
ELEVATION 70.4
HEIGHT OF FILL - FEET
SETTLEMENT-FEET
5
4
3
2
1
0
1
2
3
0
1
2
3
4
5
6
7
8
AUGUST
15
TIME IN DAYS

FIGURE 26

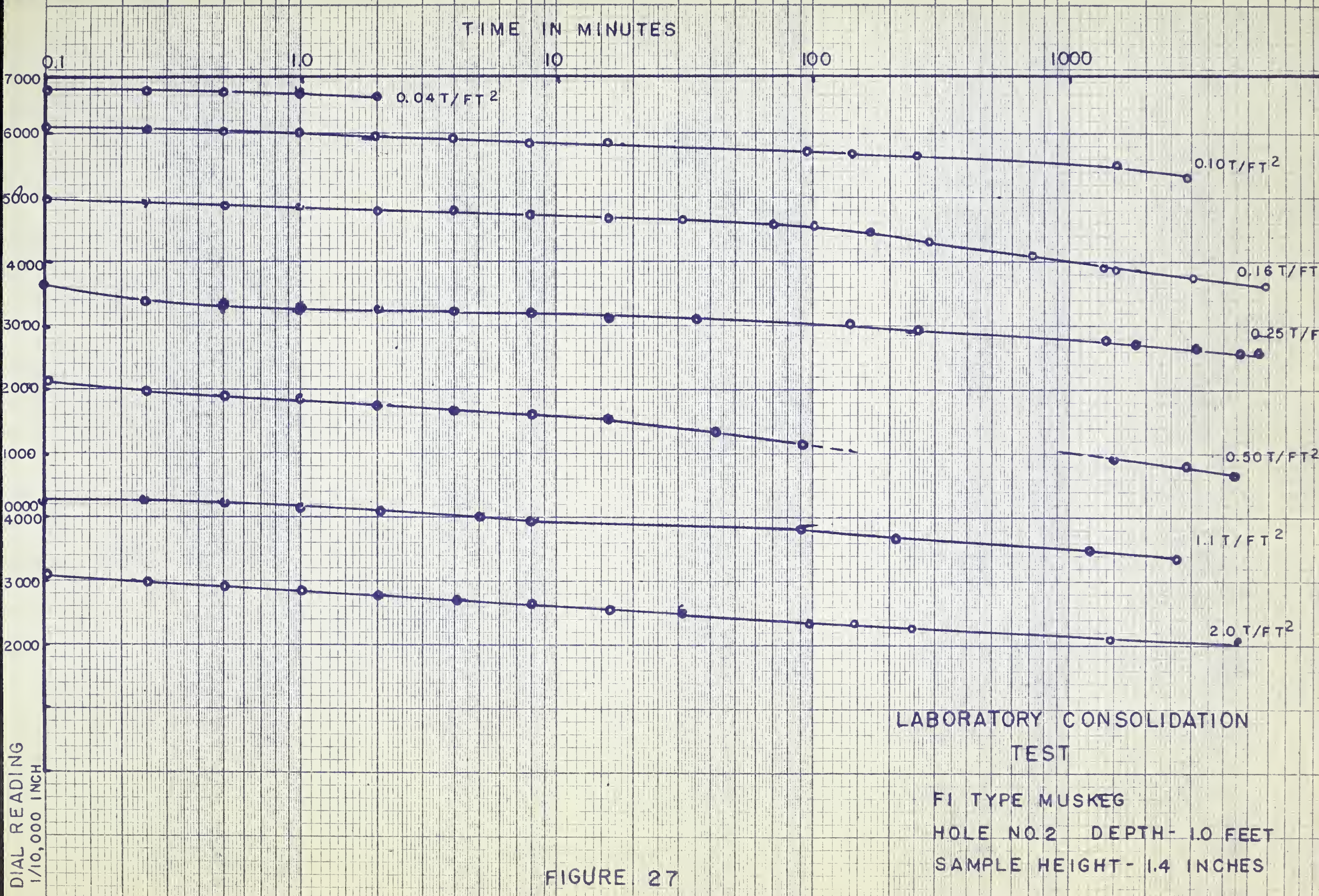

TIME IN MINUTES
0.1
1.0
10
100
1000
DIAL READING
1/10,000 INCH
7000
6000
5000
4000
3000
2000
1000
0000
4000
3000
2000
0.04 T/FT 2
0.10 T/FT 2
0.16 T/FT 2
0.25 T/FT 2
0.50 T/FT 2
1.1 T/FT 2
2.0 T/FT 2
LABORATORY CONSOLIDATION
TEST
F1 TYPE MUSKEG
HOLE NO. 2 DEPTH - 1.0 FEET
SAMPLE HEIGHT - 1.4 INCHES

FIGURE 27

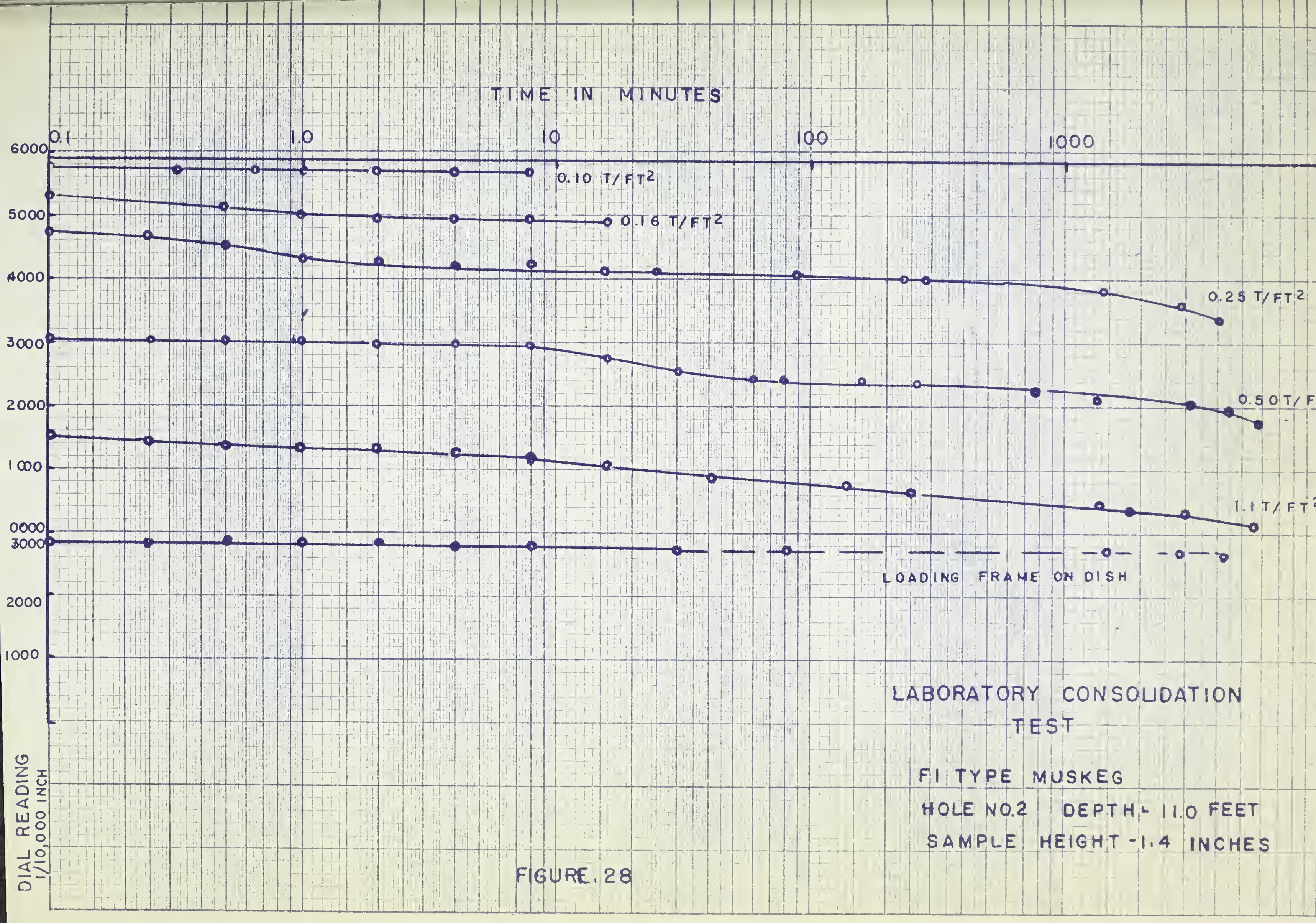

TIME IN MINUTES
0.1
1.0
10
100
1000
6000
5000
4000
3000
2000
1000
0000
3000
2000
1000
DIAL READING
1/10,000 INCH
0.10 T/FT2
0.16 T/FT2
0.25 T/FT2
0.50 T/FT2
1.1 T/FT2
LOADING FRAME ON DISH
LABORATORY CONSOLIDATION
TEST
FI TYPE MUSKEG
HOLE NO.2 DEPTH - 11.0 FEET
SAMPLE HEIGHT - 1.4 INCHES

FIGURE. 28

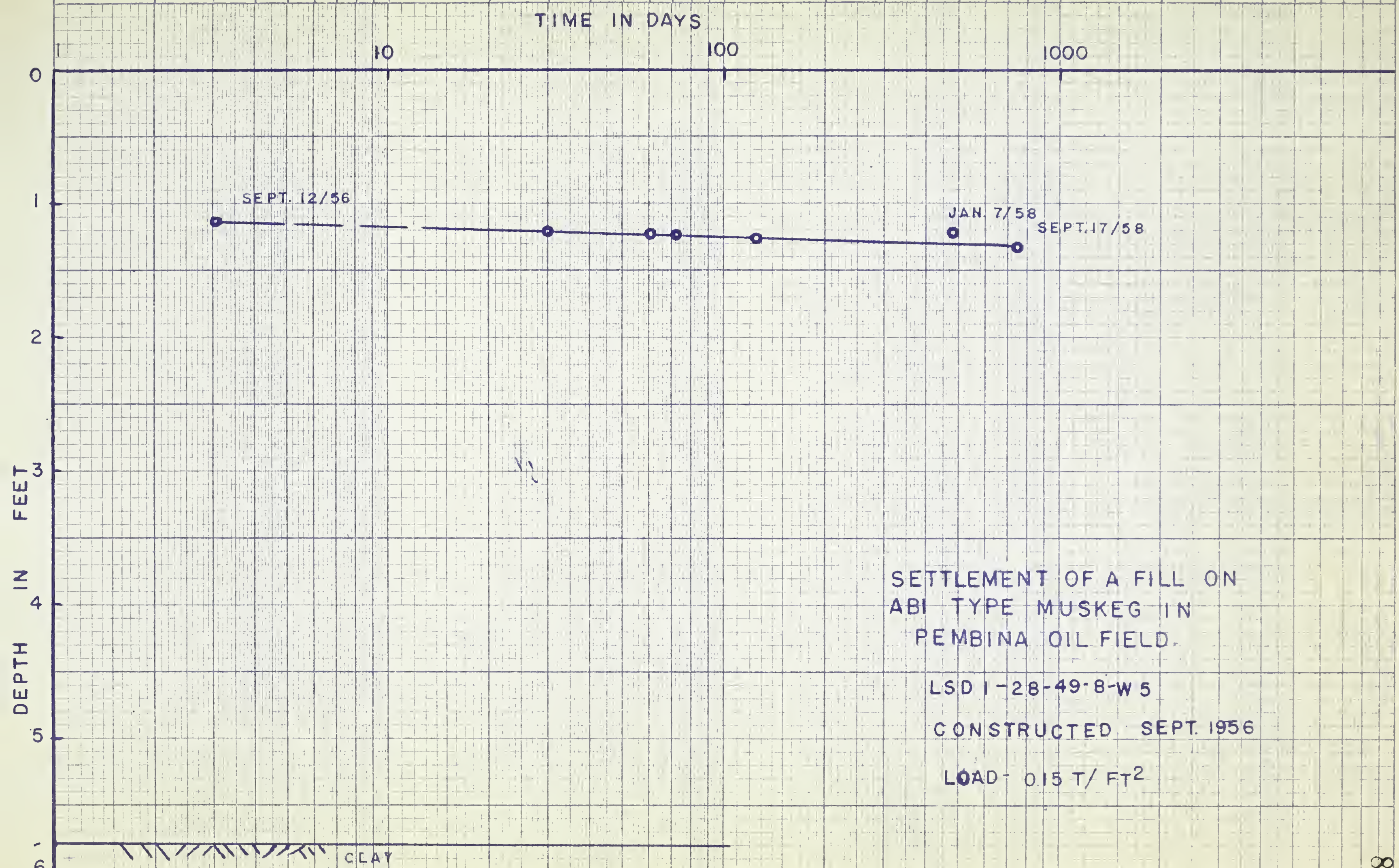

TIME IN DAYS
10
100
1000
0
1
2
3
4
5
6
DEPTH IN FEET
SEPT. 12/56
JAN. 7/58
SEPT. 17/58
SETTLEMENT OF A FILL ON ABI TYPE MUSKEG IN PEMBINA OIL FIELD.
LSD 1-28-49-8-W5
CONSTRUCTED SEPT. 1956
LOAD - 0.15 T/ FT²
CLAY

FIGURE. 29

APPENDIX III

DATA

UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W. of Alsike
LOCATION R/A Approx 700 FT N. Hwy.
HOLE #1
TECHNICIAN R.E. & Koa DATE July 9/58

Elevation of Ground Surface 70.6

Surface Coverage FI

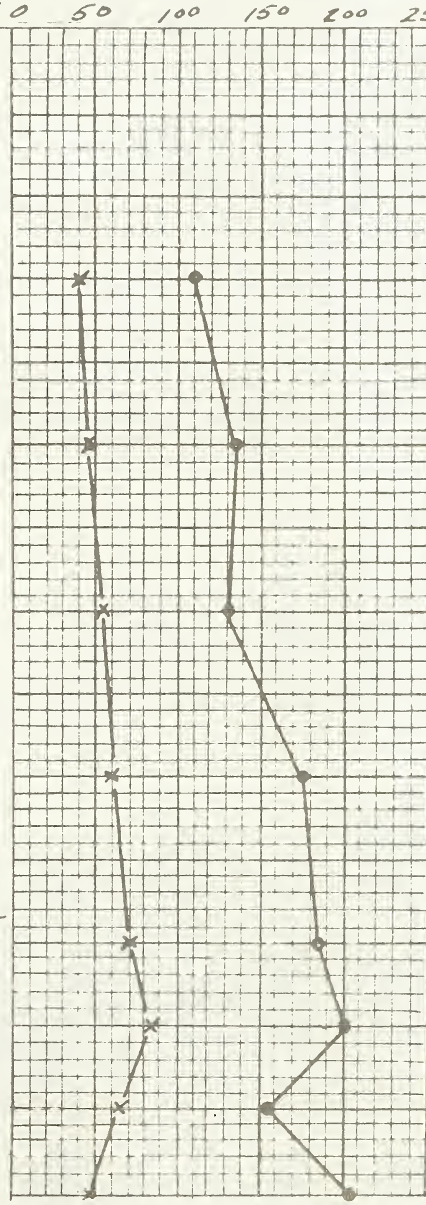
Water Conditions Water level at surface

Vane Test Method Drive vane S = 4.7T

Depth - Feet	Profile FI	15	30	45	60	75	90	105	120	135	150	S lbs/ft2
3	UD	14	14	18	22	$\boxed{24}$	20	14				110
	R	8	8									40
5	UD	12	20	25	29	28	25	18	15			135
	R	8	10	10								47
7	UD	18	25	28	28	25	25					130
	R	11	11	12	12							55
9	UD	27	33	37	37	33	31	30				175
	R	13	13	13								60
11	UD	25	32	36	39	33	32					185
	R	13	14	15	15	15						70
12	UD	26	34	40	43	34	29					200
	R	14	17	18	18	18						85
13	UD	28	33	33	32	28	28					155
	R	12	12	13	13	14	14					65
14	UD	35	44	42	39	32						205
14.5	R	9	9	9	10	10	10					47

Torque - ft lbs / Deformation - Degrees

Shear Strength - lbs/ft2



UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT *Muskeg Research*
SITE *W. of Alsike*
LOCATION *1+83 NE 4'R of ℄*
HOLE *#2*
TECHNICIAN *RE & KOM* DATE *July 9/58*

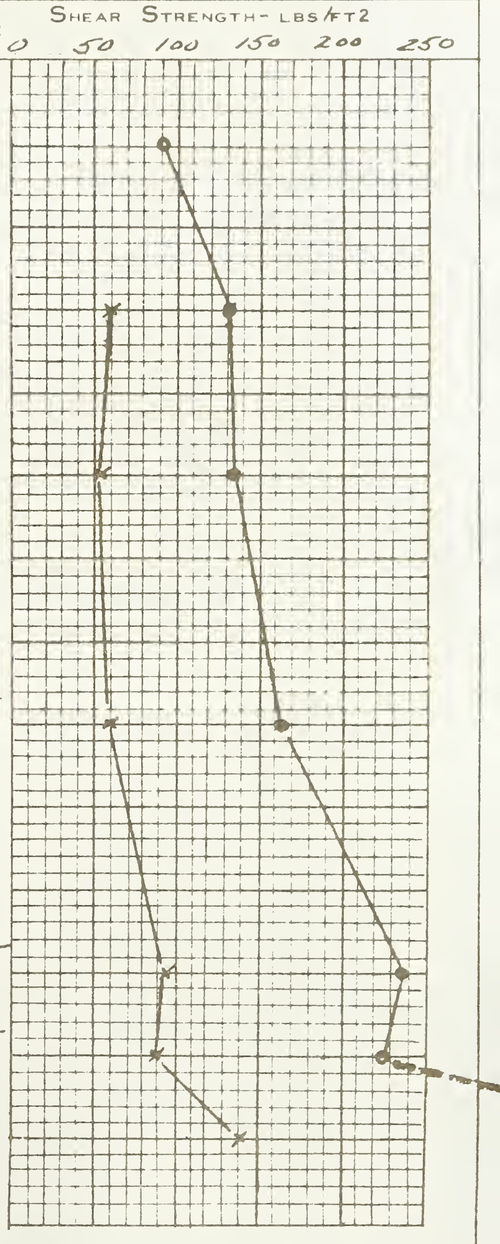
ELEVATION OF GROUND SURFACE *70±*

SURFACE COVERAGE *FI*

WATER CONDITIONS *Water level at surface*

VANE TEST METHOD *Drive vane* *S = 4.7T*

DEPTH - FEET	PROFILE ∇ FI	TORQUE - FT LBS / DEFORMATION - DEGREES 15	30	45	60	75	90	105	120	135	150	S LBS/FT2
1	UD	11	18	18	19	19	18	12				90
2												
3	UD	14	23	27	28	25	24					130
	R	11	12	13	13	13	13					60
4												
5	UD	15	21	28	29	28	22					135
		10	11	11	11							52
6												
7												
8	UD	15	24	30	33	35	31	25				165
	R	10	11	13	13	13	12					60
9												
10												
11	UD	27	38	46	50	49	40					235
	R	15	18	19	20	20	19					95
12	UD	32	44	48	41	39	34					225
	R	17	19	19	19	19						90
13	UD	30	53	81	95	120	127	130	127	120		610
	R	23	25	27	27	28	30	30	31	31		140
	Blue Clay											



UNIVERSITY OF ALBERTA
DEP T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W of Alsike
LOCATION 1+83 NE 5' R of ℄
HOLE #2 (b)
TECHNICIAN Re'KOH DATE July 9/58

ELEVATION OF GROUND SURFACE 70.6

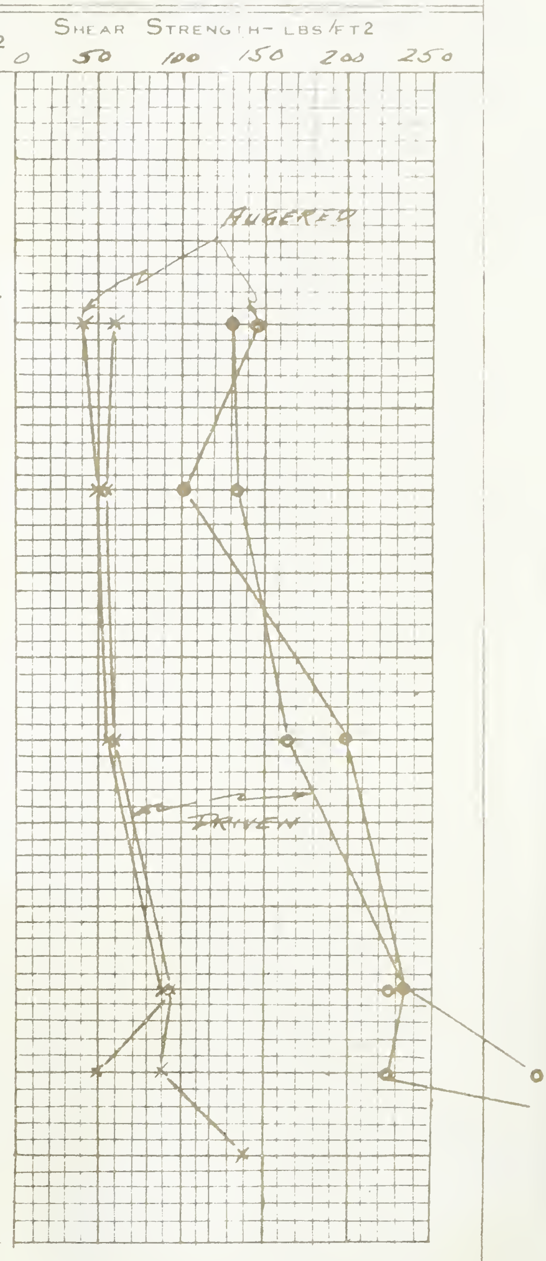
SURFACE COVERAGE FI

WATER CONDITIONS Water level at surface

VANE TEST METHOD Hand auger to half vane length from depth S=5.0T

Depth (feet)	Profile FI	TORQUE – FT LBS / DEFORMATION – DEGREES 15	30	45	60	75	90	105	120	135	150	S LBS/FT2
1												
2												
3	UD	19	24	27	29	24	20					145
	R	7	8	8	8							40
4												
5	UD	17	17	20	20	19	18					100
	R	9	9	9	10	10						50
6												
7												
8	UD	25	33	35	40	32	28					200
	R	10	11	11	11							55
9												
10												
11	UD	37	47	46	40							235
	R	17	18	18	18							90
12	UD	45	62	61	52							310
	R	9	9	10	10	10						50

Vane bottom 2" into blue clay.



UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W. of Alsike
LOCATION 2+52 NE 4'R of ℄
HOLE #3
TECHNICIAN Kelkoa DATE July 9/58

ELEVATION OF GROUND SURFACE 70^{6}

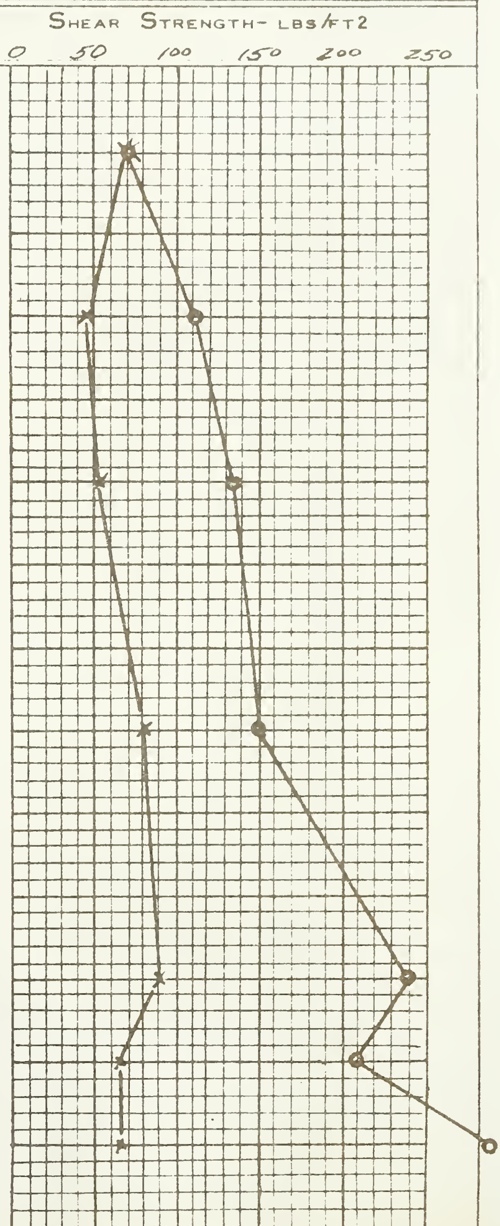
SURFACE COVERAGE F I.

WATER CONDITIONS Water level at surface.

VANE TEST METHOD Drive vane S = 4.7T

Depth – Feet	PROFILE FI	15	30	45	60	75	90	105	120	135	150	S LBS/FT2
0												
1.0	UD	10	11	11	15	15	15	11				70
	R	11	14	15	15	12	-					70
			Disturbance caused vane to drop.									
2.0												
3	UD	15	20	23	23	19	16					118
	R	9	10	11	11	11	10					47
4												
5	UD	20	27	29	29	26	22					135
	R	9	10	11	11	11	10					52
6												
7												
8	UD	28	31	32	29							150
	R	12	13	14	17	15	13					80
9												
10												
11	UD	35	46	51	49	43						240
	R	15	18	18	19	17						90
12	UD	33	40	45	39	31						210
	R	13	14	14	14	14						65
12.5	UD	25	55	62	58							290
		13	14	14	14							65
	Refusal											

TORQUE – FT LBS / DEFORMATION – DEGREES



UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W of Alsike
LOCATION Lt 72 NE 8' R of ℄
HOLE # 3 c
TECHNICIAN RL & Koa DATE July 8/58

ELEVATION OF GROUND SURFACE ?? I

SURFACE COVERAGE Ridge of BEI muskeg

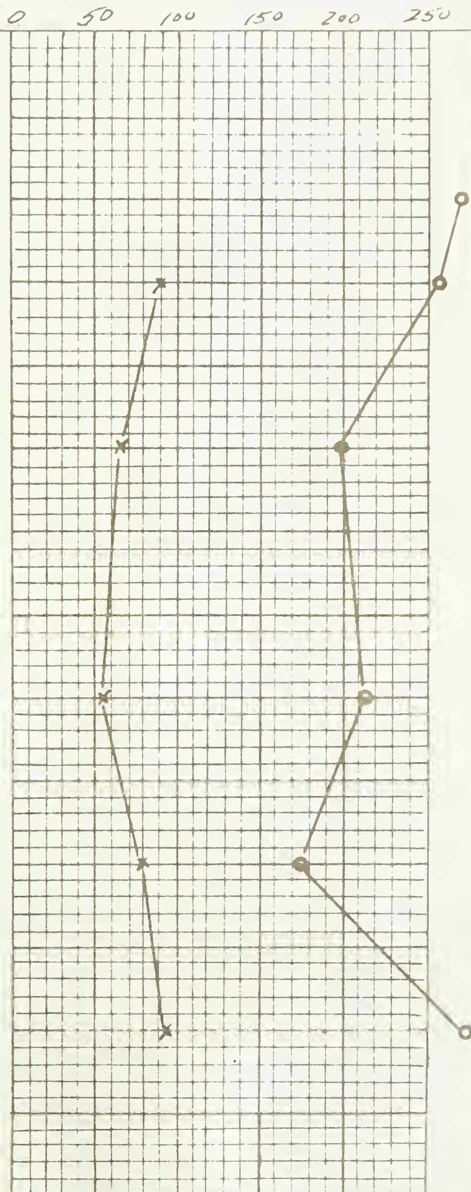
WATER CONDITIONS Water level 0.3 ft from surface of ridge

VANE TEST METHOD Drive vane. Difficult to penetrate top 2 ft due to roots
S = 4.7T

DEPTH - FEET	PROFILE		15	30	45	60	75	90	105	120	135	150	S LBS/FT2
0	BEI												
0.3	Roots.												
2		UD	20	40	60	58	51	40					280
3		UD	30	47	51	55	49	45					260
		R	17	19	17	17							90
5		UD	22	30	38	42	43	39	29				200
		R	11	12	14	14	14						65
8		UD	29	38	38	46	42	35					215
		R	11	11	12	12	12						55
10		UD	19	29	34	37	34	40					175
		R	16	16	17	17	17	17					80
12		UD	25	41	54	60	56	47					280
		R	17	20	20	20	20						95

TORQUE - FT LBS
DEFORMATION - DEGREES

SHEAR STRENGTH - LBS/FT2



UNIVERSITY OF ALBERTA DEP T OF CIVIL ENGINEERING SOIL MECHANICS LABORATORY VANE SHEAR TEST DATA	PROJECT Muskeg Research SITE W of Alsike LOCATION 2+65 NE 13' R of ℄ HOLE #4 TECHNICIAN RE & KOA DATE July 10/11/58

ELEVATION OF GROUND SURFACE 71'

SURFACE COVERAGE BEI

WATER CONDITIONS Water level 0.5 ft from surface

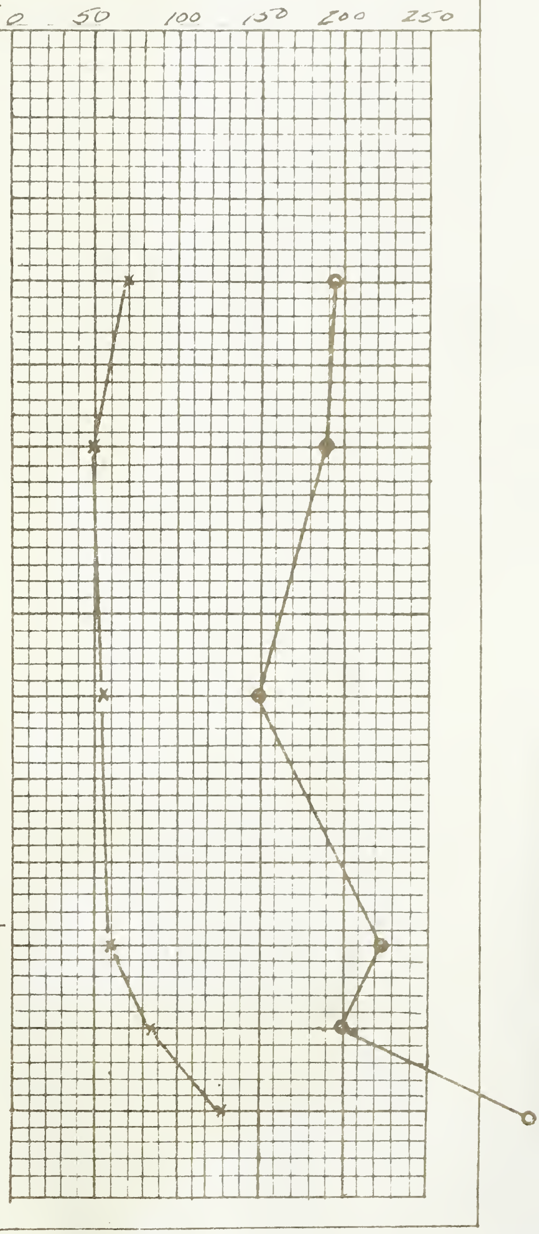
VANE TEST METHOD Auger through surface layer. Drive vane 5×4.7T

PROFILE — TORQUE-FT LBS — DEFORMATION-DEGREES — S LBS/FT2

Depth - Feet	BEI	15	30	45	60	75	90	105	120	135	150	S LBS/FT2
0.5	Roots up to 1"φ											
1												
2												
3	UD	28	39	40	42	36						195
	R	12	13	15	15	15						70
4	Rain Stopped work overnight.											
5	UD	27	33	37	40	41	30					190
	R	10	10	11	11	11						50
6												
7												
8	UD	21	29	32	32	29	23					150
	R	12	12	12	12	12						55
9												
10												
11	UD	19	30	40	48	47	38					225
	R	13	13	13	13	13						60
12	UD	39	43	37	31							200
	R	15	18	18	18	18						85
13	UD	40	66	80	78	70						375
	R	27	27	27	28	28						130

Hit refusal 4" blue clay on vane

SHEAR STRENGTH - LBS/FT2



UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W of Aksike
LOCATION 3+38 1' L of ℄
HOLE #5
TECHNICIAN RE & KOA DATE July 10/58

ELEVATION OF GROUND SURFACE 70 6

SURFACE COVERAGE FI

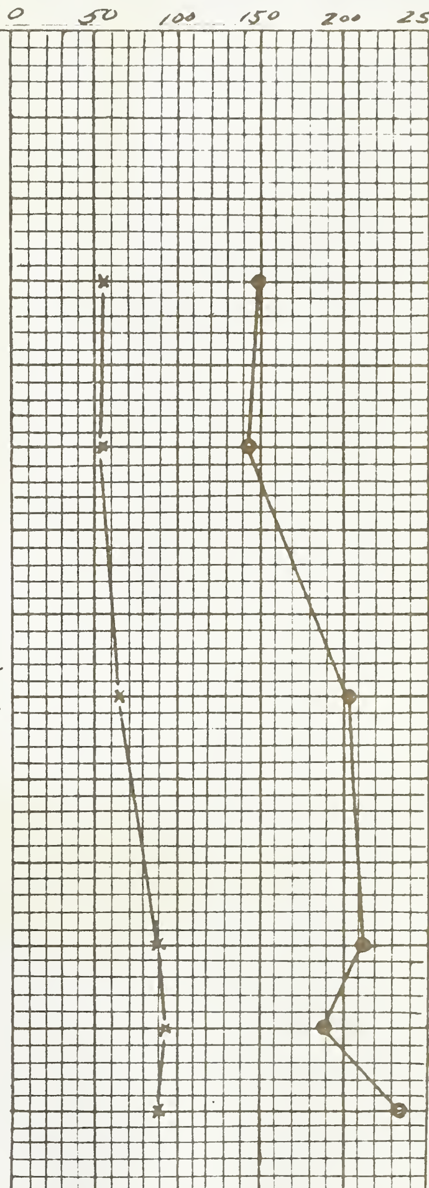
WATER CONDITIONS Water level at surface

VANE TEST METHOD Drive vane S = 4.7T

PROFILE TORQUE - FT LBS / DEFORMATION - DEGREES

DEPTH - FEET	PROFILE FI	15	30	45	60	75	90	105	120	135	150	S LBS/FT2
0												
1												
2												
3	UD	20	27	30	32	29	25					150
	R	11	12	12	12							55
4												
5	UD	18	29	30	31	30	29	27				145
	R	11	11	11	12	12	11					55
6												
7												
8	UD	20	30	35	41	44	46	30				205
	R	12	13	13	14	14	14	14				65
9												
10												
11	UD	27	39	41	45	46	40					215
	R	17	18	19	19	19	19					90
12	UD	29	40	39	31							190
	R	17	20	20	20							95
13	UD	40	50	48	48	43						235
	R	19	19	19	19							90
	Refusal											

SHEAR STRENGTH - LBS/FT2



UNIVERSITY OF ALBERTA DEP'T OF CIVIL ENGINEERING SOIL MECHANICS LABORATORY VANE SHEAR TEST DATA	PROJECT Muskeg Research SITE W of Alsike LOCATION 2+50 NE 31' L of ℄ HOLE #6 TECHNICIAN Red Koa DATE July 11/58

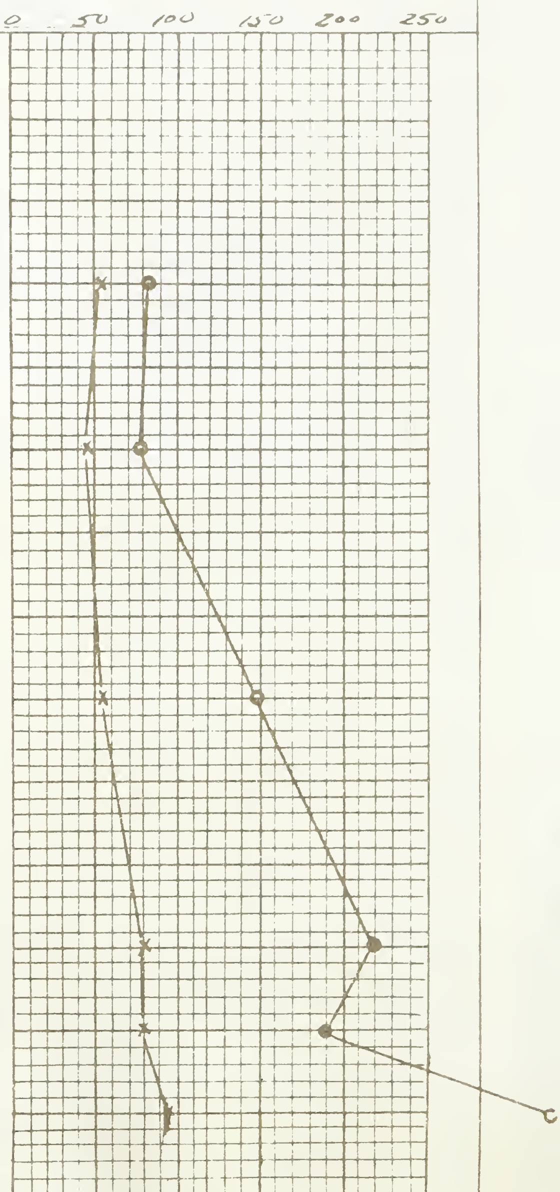
ELEVATION OF GROUND SURFACE 70'

SURFACE COVERAGE FI

WATER CONDITIONS Water level at surface

VANE TEST METHOD Drive vane S=4.7T

PROFILE FT		TORQUE-FT LBS / DEFORMATION-DEGREES 15	30	45	60	75	90	105	120	135	150	S LBS/FT2
3	UD	10	13	14	18	15	15					85
	R	11	12	12	12							55
5	UD	12	16	17	17	14						80
	R	8	9	10	10	10						47
8	UD	19	27	30	32	30	26					150
	R	9	10	11	11	11						55
	UD	21	32	40	47	46	38					220
	R	16	16	17	16	16						80
12	UD	27	39	40	40	38						190
	R	14	17	17	17							80
	UD	46	65	76	71							360
	R	20	20	20	20							95
	fusal											



UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE Wlot Alsike
LOCATION 0+91 NE C'L of E
HOLE #7
TECHNICIAN RE & RDa DATE July 15/58

ELEVATION OF GROUND SURFACE 70±

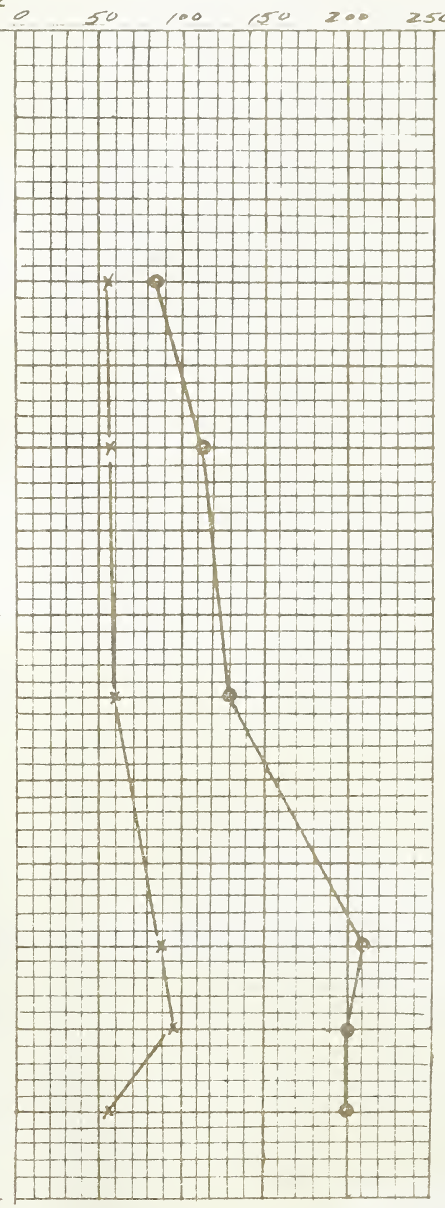
SURFACE COVERAGE FI edge

WATER CONDITIONS Water level at surface

VANE TEST METHOD Drive Vane S-47T

Depth – Feet	Profile FI	15	30	45	60	75	90	105	120	135	150	S lbs/ft2
1												
2												
3	UD	12	18	18	17	15						85
	R	11	11	12	12	11						55
4												
5	UD	18	20	21	24	22	20					115
	R	12	12	12	12	12						57
6												
7												
8	UD	18	16	28	28	27	23					130
	R	11	12	13	13	12						60
9												
10												
11	UD	25	31	40	43	45	40					210
	R	14	18	19	19	19	19					90
12	UD	28	36	41	43	42	33					200
	R	12	19	19	20	20	20					95
13	UD	41	43	40	36	35						200
	R	11	12	12	12							57

TORQUE – FT LBS
DEFORMATION – DEGREES



UNIVERSITY OF ALBERTA
DEP T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W of Alsike
LOCATION 0+09 2' L of ℄
HOLE # 8
TECHNICIAN Re E Koh DATE July 15/58

Elevation of Ground Surface 70 ±

Surface Coverage FI

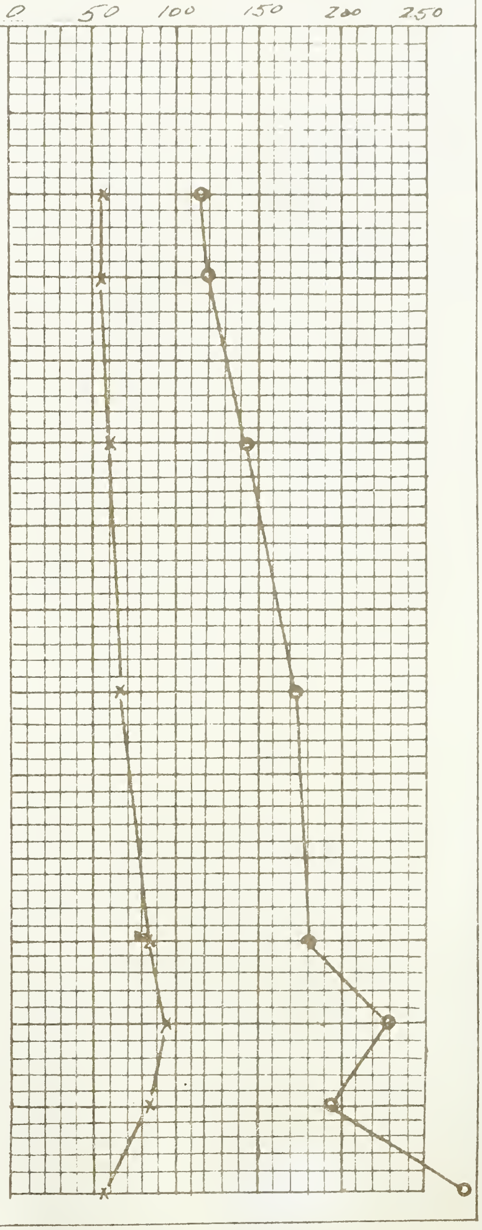
Water Conditions Water level at surface

Vane Test Method Drive vane S: 47T

Depth - Feet	Profile Fi	15	30	45	60	75	90	105	120	135	150	S lbs/ft2
1												
2	UD	11	18	24	21							115
	R	12	12	12								57
3	UD	20	25	25	24							120
	R	11	12	12	12							57
4												
5	UD	19	24	29	31	30	29					145
	R	12	12	13	13	12						60
6												
7												
8	UD	22	31	36	37	35	28					175
	R	12	14	14	14							65
9												
10												
11	UD	27	32	38	37	32						180
	R	15	15	18	15							85
12	UD	35	43	49	46							230
	R	19	20	20	20							95
13	UD	30	39	41	40	36						195
	R	14	18	18	18							85
14	UD	35	51	57	58	53						275
	R	11	11	12	12							57

TORQUE - FT LBS
DEFORMATION - DEGREES

SHEAR STRENGTH - LBS/FT2



UNIVERSITY OF ALBERTA DEP'T OF CIVIL ENGINEERING SOIL MECHANICS LABORATORY	PROJECT Muskeg Research SITE W of Alsike LOCATION 0+31 NE 9' R of E HOLE # 9
VANE SHEAR TEST DATA	TECHNICIAN Red Koh DATE July 15/58

ELEVATION OF GROUND SURFACE 71.1

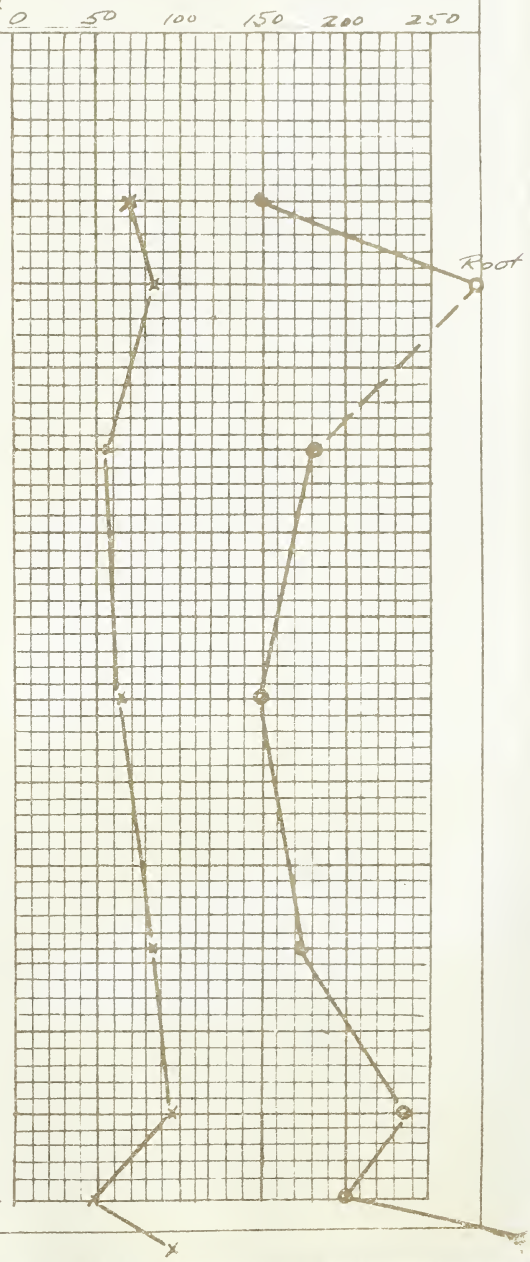
SURFACE COVERAGE EI

WATER CONDITIONS Water level 0.5 ft from surface

VANE TEST METHOD Drive vane S = 4.7T

DEPTH - FEET	PROFILE EI	TORQUE - FT LBS, DEFORMATION - DEGREES: 15	30	45	60	75	90	105	120	135	150	S LBS/FT2
0–1	Small roots											
2	UD	20	28	30	32	28						150
	R	12	15	15	15							70
3	UD	29	43	50	54	59	40	Root				280
	R	15	15	18	18	18						85
5	UD	24	30	35	38	38	32					180
	R	10	11	12	12	12						57
8	UD	19	29	31	32	32	29					150
	R	12	13	13	14	14	14					65
11	UD	20	30	33	37	35						175
	R	14	14	15	18	16						85
13	UD	21	33	41	50	50	45					235
	R	20	20	20	20							95
14	UD	30	42	39	35							200
	R	9	10	10	10							47
14.5	UD	50	70	72	72							340
	R	12	13	15	18	18						90

SHEAR STRENGTH - LBS/FT2



UNIVERSITY OF ALBERTA
DEP T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W of Alsike
LOCATION 3+92.5 6' R of ℄
HOLE #10
TECHNICIAN Zelkoa DATE July 15/58

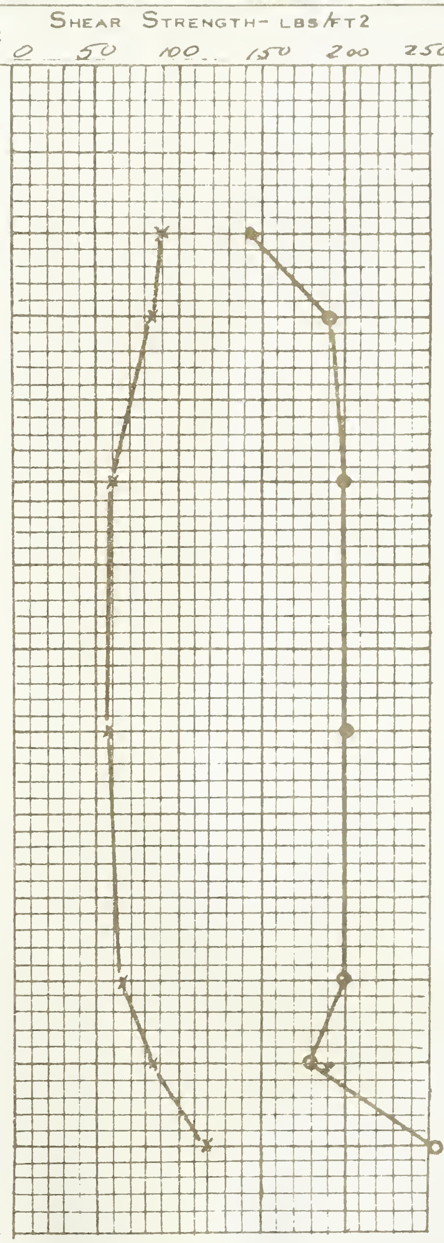
ELEVATION OF GROUND SURFACE 71'

SURFACE COVERAGE Patches of EI & FI

WATER CONDITIONS Water level 0.5 ft from surface

VANE TEST METHOD Drive vane S = 4.7T.

DEPTH – FEET	PROFILE	TORQUE – FT LBS / DEFORMATION – DEGREES 15	30	45	60	75	90	105	120	135	150	S LBS/FT2
0	ET											
	Roots 1/2"±											
2	UD	20	27	31	31	30						145
	R	19	19	19								90
3	UD	28	37	40	40	35						190
	R	16	16	15	18	17						85
5	UD	28	31	33	40	42	28					200
	R	12	13	13	13							60
8	UD	25	32	40	42	40						200
	R	11	11	12	12	12						57
11	UD	31	37	42	40							200
	R	12	13	14	14	13						65
12	UD	24	31	38	35							180
	R	14	17	18	18							85
13	UD	35	48	54	52							255
	R	24	25	25	22							120



UNIVERSITY OF ALBERTA
DEP T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W of Aksike
LOCATION 1+00 S 6' R of ℄
HOLE #11
TECHNICIAN RE & KOA DATE July 16/58

ELEVATION OF GROUND SURFACE 70 6

SURFACE COVERAGE Fl patch amidst El

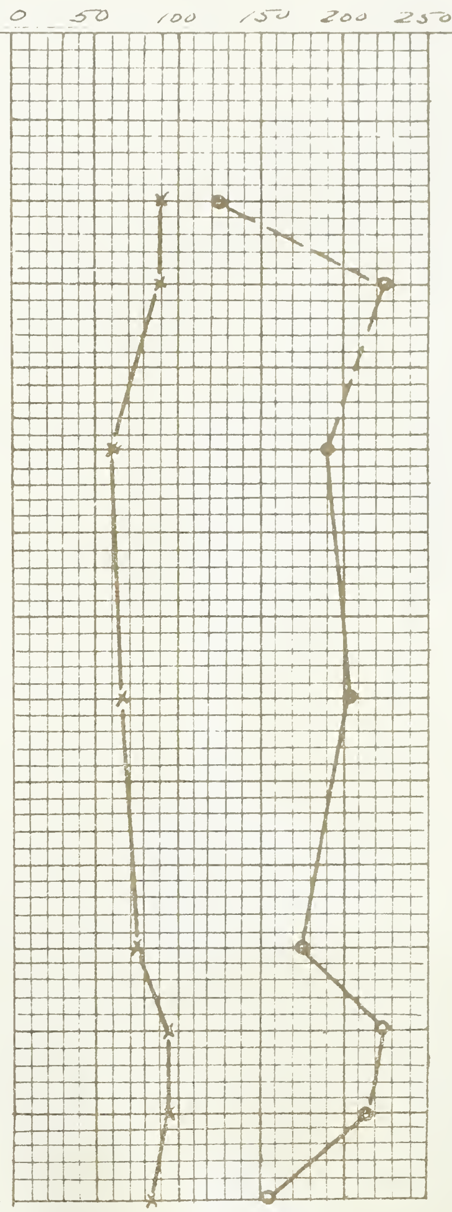
WATER CONDITIONS Water level at surface

VANE TEST METHOD Drive vane S=4.7T

Depth – Feet	Profile		15	30	45	60	75	90	105	120	135	150	S lbs/ft2
0	Fl												
1	small Roots												
2		UD	31	37	36	33							175
		R	17	19	18	15							90
3		UD	28	40	48	48	47	45 Root					225
		R	14	15	15	19	19	19					90
4													
5		UD	26	32	39	40	41	33					190
		R	11	13	13	13							60
6													
7													
8		UD	28	35	42	44	41						205
		R	11	12	13	14	13						65
9													
10													
11		UD	27	31	35	37	33						175
		R	15	16	16	16	16						75
12		UD	26	38	47	48	44						225
		R	19	20	20	20							95
13		UD	30	40	46	41	38						215
		R	17	19	20	20							95
14		UD	30	32	33	31							155
		R	14	16	18	18	18						85

TORQUE – FT LBS
DEFORMATION – DEGREES

SHEAR STRENGTH – LBS/FT2



UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE IN of Alsike
LOCATION 1+79 S 3' L of ℄
HOLE #12
TECHNICIAN RE & KOA DATE July 16/58

Elevation of Ground Surface 70.6

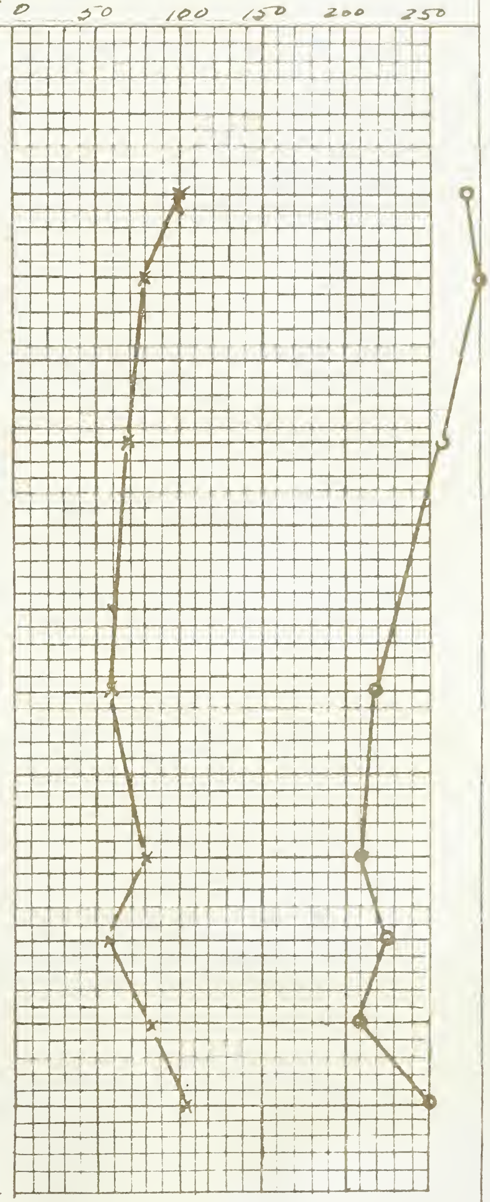
Surface Coverage clear patch of I amidst A growth

Water Conditions Water level at surface

Vane Test Method Drive vane S=47T

Depth - Feet	Profile		Torque - ft lbs, Deformation - Degrees 15	30	45	60	75	90	105	120	135	150	S lbs/ft2
0	AI												
	Small roots												
1													
						Root							
2		UD	34	45	58	58	53						275
		R	20	21	20	19							100
		UD	41	49	56	60	55						280
3		R	13	14	14	16	17	16					80
4													
5		UD	34	44	52	55	52						260
		R	12	13	14	15	15	14					70
6													
7													
8		UD	23	31	42	47	40						220
		R	12	12	13	13	13	13					60
9													
10		UD	30	39	45	42							210
		R	15	16	16	17	17	16					80
11		UD	26	36	47	48	44						225
		R	10	11	12	11							57
12		UD	30	40	45	42							210
		R	15	16	18	18	18						85
13		UD	40	52	53	57							250
		R	18	20	20	22	21						105
	Refusal												

Shear Strength - lbs/ft2



UNIVERSITY OF ALBERTA
DEP T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W of Alsike
LOCATION 2+425 4' L of ℄
HOLE #13
TECHNICIAN RE & KOA DATE July 16/58

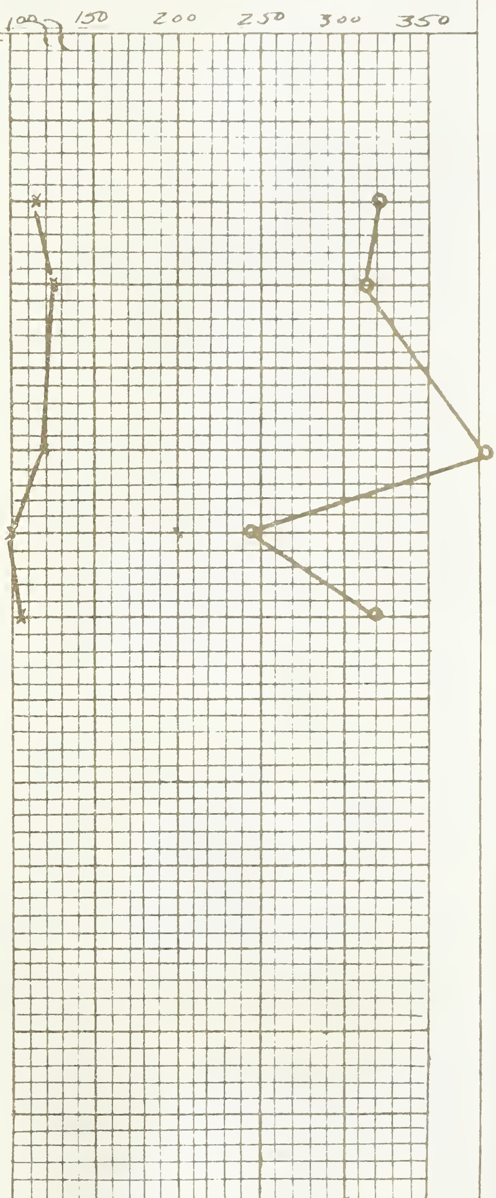
ELEVATION OF GROUND SURFACE 71.0

SURFACE COVERAGE Patchs of EI amidst AEI

WATER CONDITIONS Water level 0.3 ft from surface.

VANE TEST METHOD Drive vane.

DEPTH - FEET	PROFILE	TORQUE - FT LBS / DEFORMATION - DEGREES 15	30	45	60	75	90	105	120	135	150	S LBS/FT2
0	EI											
1												
						Root						
2	UD	40	53	64	68	66						320
	R	19	22	24	25	25						115
3	UD	48	58	62	64	67	54					315
	R	20	18	18	20	27	27					125
4												
5	UD	40	59	64	72	85	80					400
	R	20	21	23	26	26	26					120
6	UD	34	48	52	51							245
	R	19	20	21	20							100
7	UD	47	57	60	68	68	65					320
	R Refusal	20	21	22	22							105



UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

VANE SHEAR TEST DATA

PROJECT Muskeg Research
SITE W of Alsike
LOCATION 2+16 NE 26'R of ℄
HOLE #14
TECHNICIAN Koa DATE Sept 11/58

Elevation of Ground Surface ______

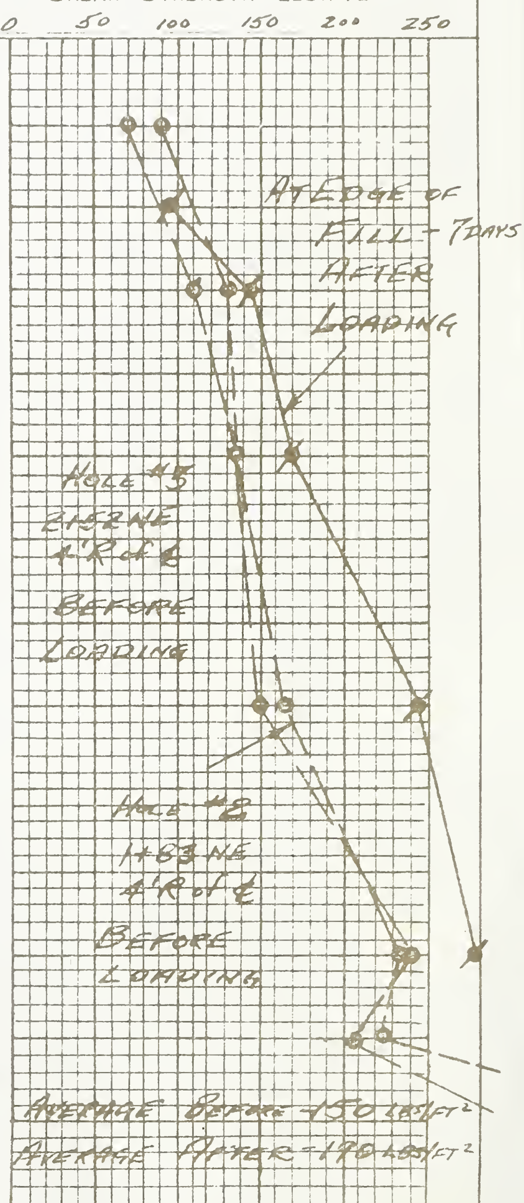
Surface Coverage FI - tested to side of fill after failure occurred

Water Conditions Approximately 0.5 ft below surface. Mat appeared dry

Vane Test Method Drive vane . S=4.7T

Depth – Feet	Profile	15	30	45	60	75	90	105	120	135	150	S lbs/ft2
2	UD	10	17	19	20	19	18					95
	R	10	11	12	11	11						65
3	UD	19	25	29	31	29	24					145
	R	11	12	13	11							60
5	UD	20	28	31	37	39	37	30				170
	R	12	14	15	14	10						70
8	UD	20	38	45	50	52	45					245
	R	19	20	20	18							95
11	UD	30	48	58	60	55						280
	R	20	22	26	26	25						120

Torque – ft lbs / Deformation – Degrees



UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

MOISTURE AND ASH CONTENT

PROJECT Muskeg Research
SITE W of Alsike
LOCATION 700'N on R/A , 153NE.
HOLE No 1 & 2
TECHNICIAN Koa DATE July 21/58

Hole No.	1	1	1	1	1	1
Depth	1'0"	1'0"	5'0"	7'0"	10'0'	10.5'
Sample No.	D-1 (packed)	D-1 (loose)	D (loose)	—	—	From Shelby Tube
Wt. Sample Wt. + Tare	190.11	189.22	180.22	151.04	159.86	179.25
Wt. Sample Dry + Tare	51.35	50.10	50.95	47.15	52.61	51.06
Wt. Water	148.76	139.12	129.27	103.89	107.25	128.19
Wt. Ash + Tare	—	—	—	—	—	—
Tare	40.10	40.20	40.60	39.40	40.65	40.48
Wt. Dry Soil	11.25	9.90	10.35	7.75	11.96	10.58
Wt. Ash	—	—	—	—	—	—
Moisture Content %	1320	1400	1250	1340	900	1220
Ash Content - %			—	—	—	—
S- undisturbed		110	135	130	185	
S- remoulded		40	47	55	70	

Hole No.	2	2	2	2	2	2
Depth	1'	3'	5'	8'	11'	13'
Sample No.	2-1-1	2-2-3	2-3-5	2-4-8	2-5-11	2-6-13
Wt. Sample Wet + Tare	172.43	143.15	136.71	163.91	149.68	164.42
Wt. Sample Dry + Tare	49.82	50.53	49.89	52.52	50.88	137.01
Wt. Water	122.61	92.62	86.82	111.39	98.80	27.41
Wt. Ash + Tare	41.67	42.62	40.94	42.61	41.58	135.39
Tare	40.05	39.83	39.28	40.41	38.84	39.55
Wt. Dry Soil	9.77	10.70	10.61	12.11	12.04	97.46
Wt. Ash	1.62	2.79	1.66	2.20	2.74	96.84
Moisture Content %	1250	865	820	920	820	28.1
Ash Content %	16.6	26.0	15.6	18.1	22.7	99.0
S- undisturbed	90	130	135	165	235	
S- remoulded	—	60	52	60	95	

Remarks: Hole No. 1 - F1 muskeg
No. 2 - F1 edge

UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

MOISTURE AND ASH CONTENT

PROJECT Muskeg Research
SITE W of Aksike
LOCATION 2+65 NE 3+38 NE
HOLE #4 & 5
TECHNICIAN RE & KOA DATE July 21/58

Hole No.	4	4	4	4	4	
Depth	1'	3'	5'	8'	11'	
Sample No.	4-1-1	4-2-3	4-3-5	4-4-8	4-5-11	
Wt. Sample Wt. +Tare	166.70	191.28	140.74	148.02	150.15	
Wt. Sample Dry +Tare	52.40	53.90	48.00	49.80	52.47	
Wt. Water	114.30	137.38	92.74	98.22	97.68	
Wt. Ash +Tare	40.70	42.90	41.42	41.13	45.43	
Tare	38.72	39.48	39.88	39.59	41.10	
Wt. Dry Soil	13.68	14.42	9.12	10.21	11.37	
Wt. Ash	1.98	3.42	1.54	1.54	4.33	
Moisture Content %	840	950	1010	960	860	
Ash Content - %	14.5	23.7	16.9	15.1	38.0	
S- undisturbed	—	195	190	150	225	
S- remoulded	—	70	50	55	60	
Hole No.	5	5	5	5	5	
Depth	1'	3'	5'	8'	11'	
Sample No.	5-1-1	5-2-3	5-3-5	5-4-8	5-5-11	
Wt. Sample Wet+Tare	151.30	180.89	175.93	145.14	149.76	
Wt. Sample Dry+Tare	47.35 ~~[illegible]~~	51.29	49.34	48.38	51.16	
Wt. Water	103.95	129.60	126.59	96.76	98.60	
Wt. Ash+ Tare	40.71	41.30	41.39	41.47	40.79	
Tare	39.22	39.37	39.81	39.46	38.63	
Wt. Dry Soil	8.13	11.92	9.53	8.92	12.53	
Wt. Ash	1.49	1.93	1.58	2.01	2.16	
Moisture Content %	1280	1090	1320	1080	790	
Ash Content %	18.3	16.2	16.6	22.5	17.2	
S- undisturbed	—	150	145	205	215	
S- remoulded	—	55	55	65	90	

Remarks #4 - BEI
#5 - FI

UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

MOISTURE AND ASH CONTENT

PROJECT Muskeg Research
SITE W of Alsike
LOCATION 2+50 NE 0+91 NE
HOLE #6 , #7
TECHNICIAN RE & KOA DATE July 21/58

Hole No.	6	6	6	6	6	6
Depth	1'	3'	5'	8'	11'	12.5'
Sample No.	6-1-1	6-2-3	6-3-5	6-4-8	6-5-11	6-6-12.5
Wt. Sample Wt. + Tare	217.51	186.14	141.56	166.44	147.21	201.43
Wt. Sample Dry + Tare	50.08	55.54	47.63	49.58	50.59	126.87
Wt. Water	167.43	130.60	93.93	116.86	96.62	74.56
Wt. Ash + Tare	41.84	41.09	40.55	41.16	42.17	—
Tare	40.07	39.35	39.23	38.82	40.21	40.10
Wt. Dry Soil	10.01	15.19	8.40	10.76	10.38	86.77
Wt. Ash	1.77	1.74	1.32	2.34	1.96	—
Moisture Content %	1660	860	1110	1090	930	86
Ash Content - %	17.6	11.5	14.7	21.7	18.9	
S- undisturbed	—	85	80	150	220	-
S- remoulded	—	55	47	55	80	-

Hole No.	7	7	7	7	7	
Depth	1'	3'	5'	8'	11'	
Sample No.	7-1-1	7-2-3	7-3-5	7-4-8	7-5-11	
Wt. Sample Wet + Tare	166.78	147.23	169.35	166.58	169.00	
Wt. Sample Dry + Tare	48.06	50.20	48.85	49.94	51.75	
Wt. Water	118.72	97.03	120.50	116.64	117.25	
Wt. Ash + Tare	41.41	42.47	41.91	42.20	43.09	
Tare	39.46	39.97	39.66	40.15	40.35	
Wt. Dry Soil	8.60	10.23	9.19	9.79	11.40	
Wt. Ash	1.95	2.50	2.25	2.05	2.74	
Moisture Content %	1380	950	1310	1190	1030	
Ash Content %	22.7	24.3	24.5	21.0	24.1	
S- undisturbed		85	115	130	210	
S- remoulded		55	57	60	90	

Remarks #6 - FI
#7 - FI edge

UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

MOISTURE AND ASH CONTENT

PROJECT Muskeg Research
SITE W of Alsike
LOCATION 2+09 S , 0+31 NE
HOLE #8 , #9
TECHNICIAN Relkoh DATE July 21/58

Hole No.	8	8	8	8	8	
Depth	1'	3'	5'	8'	11'	
Sample No.	8-1-1	8-2-3	8-3-5	8-4-8	8-5-11	
Wt. Sample Wt. +Tare	157.52	165.16	142.86	173.31	139.03	
Wt. Sample Dry +Tare	50.15	50.15	47.01	50.68	48.74	
Wt. Water	107.37	115.01	95.85	122.63	90.29	
Wt. Ash +Tare	41.84	42.31	41.74	42.83	44.10	
Tare	40.15	40.03	39.83	40.00	39.80	
Wt. Dry Soil	10.00	10.12	7.18	10.68	8.94	
Wt. Ash	1.69	2.19	1.91	2.83	4.30	
Moisture Content %	1070	1130	1330	1150	1010	
Ash Content- %	16.9	21.6	26.7	26.5	48.0	
S- undisturbed		120	145	175	180	
S- remoulded		57	60	65	85	
Hole No.	9	9	9	9	9	9
Depth	1'	3'	5'	8'	11'	13
Sample No.	9-1-1	9-2-3	9-3-5	9-4-8	9-5-11	9-6-13
Wt. Sample Wet+Tare	218.91	173.10	162.64	151.70	144.89	142.40
Wt. Sample Dry+Tare	55.63	55.94	52.30	50.33	51.52	53.45
Wt. Water	163.28	117.16	110.34	101.37	93.37	88.95
Wt. Ash+ Tare	41.95	42.83	42.14	41.60	42.27	42.82
Tare	39.08	40.81	40.40	39.94	40.70	39.07
Wt. Dry Soil	16.55	15.13	11.90	10.39	10.82	14.38
Wt. Ash	2.87	2.02	1.74	1.66	1.57	3.75
Moisture Content %	990	780	930	980	860	620
Ash Content %	17.3	13.3	14.6	16.0	14.5	25.9
S- undisturbed	—	280	180	150	175	235
S- remoulded	—	85	57	65	85	95

Remarks # 8 - FI
9 - EI

UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

MOISTURE AND ASH CONTENT

PROJECT Muskeg Research
SITE 1K' of Alsike
LOCATION 0+92 S , 1+00 S
HOLE #10 , #11
TECHNICIAN RE L KoA DATE July 21/58

Hole No.	10	10	10	10	10	10
Depth	1'	3'	5'	8'	11'	12'
Sample No.	10-1-1	10-2-3	10-3-5	10-4-8	10-5-11	10-6-12
Wt. Sample Wt. + Tare	193.94	143.90	158.19	157.77	178.78	163.87
Wt. Sample Dry + Tare	53.46	51.83	51.68	49.90	69.45	65.36
Wt. Water	140.48	92.07	106.51	107.87	109.33	98.51
Wt. Ash + Tare	41.81	42.61	41.82	41.64	42.25	53.96
Tare	39.60	40.61	39.08	39.20	39.74	50.83
Wt. Dry Soil	13.86	11.22	12.60	10.70	9.71	14.53
Wt. Ash	2.21	2.00	2.74	2.44	2.51	3.13
Moisture Content %	1010	820	850	1010	1120	680
Ash Content - %	15.9	17.8	21.7	22.8	25.8	21.6
S- undisturbed	—	190	200	200	200	180
S- remoulded	—	85	60	57	65	85
Hole No.	11	11	11	11	11	
Depth	1'	3'	5'	8'	11'	
Sample No.	11-1-1	11-2-3	11-3-5	11-4-8	11-5-11	
Wt. Sample Wet + Tare	166.89	166.90	155.36	172.48	167.12	
Wt. Sample Dry + Tare	49.10	55.83	50.97	52.46	53.72	
Wt. Water	117.79	111.07	104.39	120.02	113.40	
Wt. Ash + Tare	40.58	41.83	42.12	41.50	42.56	
Tare	39.15	39.93	40.25	39.43	40.37	
Wt. Dry Soil	9.95	10.90	10.72	13.03	13.35	
Wt. Ash	1.43	1.90	1.87	2.07	2.19	
Moisture Content %	1190	1010	970	920	850	
Ash Content %	14.4	17.4	17.4	15.8	15.1	
S- undisturbed	—	225	190	205	175	
S- remoulded	—	90	60	65	75	

Remarks #10 - Patches of EI and FI
#11 - FI amidst EI

UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

MOISTURE AND ASH CONTENT

PROJECT Muskeg Research
SITE W of [illegible]
LOCATION [illegible]
HOLE # 12
TECHNICIAN [illegible] DATE July 21 58

Hole No.	12	12	12	12	12	
Depth	1'	2'	5'	8'	10'	
Sample No.	12-1-1	12-2-3	12-3-5	12-4-8	12-5-10	
Wt Sample Wt +Tare	178.77	168.95	140.75	174.04	184.54	
Wt. Sample Dry +Tare	59.35	53.01	50.23	55.23	54.72	
Wt. Water	119.42	115.94	90.52	118.81	129.82	
Wt. Ash +Tare	42.77	41.43	41.43	47.78	42.00	
Tare	40.47	39.88	39.78	39.10	40.10	
Wt. Dry Soil	18.88	13.33	10.45	16.13	14.62	
Wt. Ash	2.30	1.75	1.98	3.68	1.90	
Moisture Content %	1350	870	870	740	890	
Ash Content - %	25.9	13.1	19.0	22.8	13.0	
S- undisturbed	—	280	260	220	210	
S- remoulded	—	80	70	60	80	
Hole No.						
Depth						
Sample No.						
Wt. Sample Wet+Tare						
Wt. Sample Dry+Tare						
Wt. Water						
Wt. Ash+ Tare						
Tare						
Wt. Dry Soil						
Wt. Ash						
Moisture Content %						
Ash Content %						
S- undisturbed						
S- remoulded						

Remarks #12 - I amidst A growth

UNIVERSITY OF ALBERTA DEP'T. OF CIVIL ENGINEERING Soil Mechanics Laboratory FIELD DENSITY TEST	PROJECT MUSKEG RESEARCH SITE NW of ALSIKE SAMPLE LOCATION TRANSITION SECTION HOLE #1 DEPTH. 0-4" TECHNICIAN KOH DATE OCT 25/58

VOLUME OF HOLE	SOILTEST	WEIGHT (GMS.)
(a) ~~Sand & container before test~~	VOLUMETER	FINAL RDG = 0.0328
(b) ~~Sand & container after test~~		INITIAL RDG = 0.0090
(c) ~~Sand used in test (a - b - s)~~		VOLUME HOLE = 0.0238 FT³
(d) ~~Volume of hole (c ÷ r)~~		Cu. ~~cm.~~

WET DENSITY		
(e) Wet soil & container	1545.8 gms	
(f) Container No. 2	178.0	
(g) Wet soil (e - f)	1367.8	
(h) Wet density (g ÷ d) x 62.4	126.8	lbs/ cu. ft.

MOISTURE CONTENT	
(j) Water + soil + container	1545.8
(k) Soil + container	1415.1
(m) Water (j - k)	130.7
(n) Container No. 2	178.0
(o) Soil (k - n)	1284.0

(p) MOISTURE CONTENT (m ÷ o) x 100 = 10.2 %

(q) DRY DENSITY (h ÷ (100 + p)) x 100 = 114.8 lbs/ cu. ft.

(r) ~~Unit weight of calibrated sand~~ = ______ gms/ cu. cm.

(s) ~~Wt. of sand in funnel and hole of density pan~~ = ______ gms.

Compaction Equipment TRUCKS & D-6 CAT

General Soil Type SILTY SAND

Remarks:

Engineer

Tested by KOH

UNIVERSITY OF ALBERTA DEP'T. OF CIVIL ENGINEERING Soil Mechanics Laboratory FIELD DENSITY TEST	
PROJECT	MUSKEG RESEARCH
SITE	W of ALSIKE
SAMPLE	
LOCATION	TRANSITION SECTION
HOLE #2	DEPTH. 0"-4"
TECHNICIAN Koy	DATE Oct 25/58

VOLUME OF HOLE	SOILTEST VOLUMETER	WEIGHT (GMS.)
~~(a) Sand & container before test~~	FINAL RDG - 0.0324	
~~(b) Sand & container after test~~	INITIAL RDG - 0.0090	
~~(c) Sand used in test (a - b - s)~~	VOLUME OF HOLE - 0.0234 ft^3	
~~(d) Volume of hole (c ÷ r)~~		~~Cu. cm.~~

WET DENSITY		
(e) Wet soil & container	1523.2	
(f) Container No. 1	183.5	
(g) Wet soil (e - f)	1339.7	
(h) Wet density (g ÷ d) x 62.4	126.2	lbs/ cu. ft.

MOISTURE CONTENT	
(j) Water + soil + container	1523.2
(k) Soil + container	1399.8
(m) Water (j - k)	123.4
(n) Container No. 1	183.5
(o) Soil (k - n)	1216.3

(p) MOISTURE CONTENT (m ÷ o) x 100 = 10.1 %

(q) DRY DENSITY (h ÷ (100 + p)) x 100 = 114.4 lbs/ cu. ft.

~~(r) Unit weight of calibrated sand = ______ gms/ cu. cm.~~

~~(s) Wt. of sand in funnel and hole of density pan = ______ gms.~~

Compaction Equipment TRUCKS & D-6 CAT

General Soil Type SILTY SAND

Remarks: ______

Engineer ______

Tested by Koy

UNIVERSITY of ALBERTA
DEP'T of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
TEST HOLE LOG

PROJECT Muskeg Research
SITE W. of Alsike
SAMPLE
LOCATION 1+83 NE - 3' R of ℄
HOLE #2 DEPTH 12.6
TECHNICIAN Redkoa DATE July 9/58

Elevation of Ground Surface (Zero Depth) 70.6
Method of Advancing Hole Hand Auger
Remarks: FI Muskeg - 3" Shelby Tube Samples and Moisture Content Samples Taken.

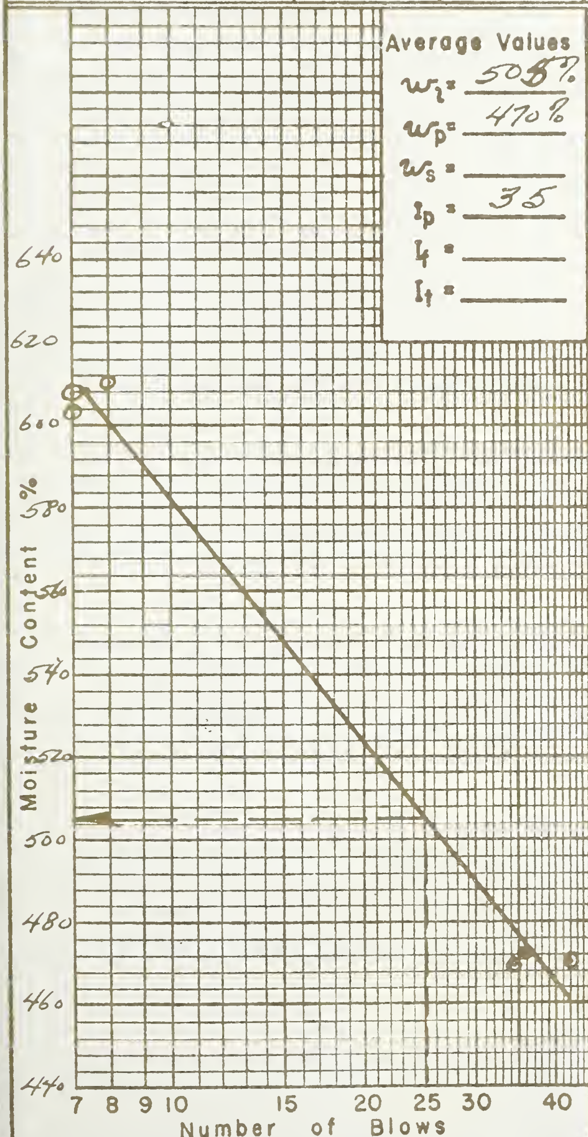
Depth in Feet	Field Classification	Sample No.	Sample Depth
0 (FI.)	Shelby tube driven to 2' - Recovery approx 6".	#2 40	0'-2'
1	Organic terrain FI.	2-1	1'-0"
	Fibrous peat.		
3		2-2	3'-0"
	Tube driven Recovery -3" Not kept.		
5	Fibrous peat.	2-3	5'-0"
8		2-4	8'-0"
10	Dark brown peat well decomposed.	2-5	11'-0"
11	Tube sample		
12.6	Blue clay.		

UNIVERSITY of ALBERTA
DEP'T· of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT Muskeg Research
SITE W of Alsike
SAMPLE Shelby Tube
LOCATION 1+83 NE – 3' R of ℄
HOLE #2 DEPTH 11.0 FT.
TECHNICIAN Kon. DATE Dec 31/58

Liquid Limit

Trial No.	1	2	3	4	5	6
No. of Blows	7	8	7	43	36	34
Container No.	V-83	V-82	V-84	V-71	V-81	V-85
Wt. Sample Wet + Tare	92.6204	112.2667	90.6865	98.4535	81.1887	93.4298
Wt. Sample Dry + Tare	80.1470	94.8770	75.9082	84.3907	69.7469	82.6954
Wt. Water	12.4734	17.3897	14.7783	14.0628	11.4418	10.7344
Tare Container	78.0936	92.0310	73.4652	81.3762	67.3238	80.4065
Wt. of Dry Soil	2.0534	2.8460	2.4430	3.0145	2.4231	2.2889
Moisture Content w%	607	610	605	467	473	469



Average Values

w_l = 505%
w_p = 470%
w_s =
I_p = 35
I_f =
I_t =

Plastic Limit

Trial No.	1	2	3
Container No.	1	4	5
Wt. Sample Wet + Tare	37.0910	41.5376	37.1072
Wt. Sample Dry + Tare	35.5923	40.1230	35.6862
Wt. Water	1.4987	1.4146	1.4210
Tare Container	35.2648	39.8180	35.3962
Wt. of Dry Soil	0.3275	0.3050	0.2900
Moisture Content %	458	465	490

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:
Organic peat.
11 ft Depth.
Well decomposed.

Remarks:
Difficult to
roll to 1/8" for
Plastic Limit test

UNIVERSITY of ALBERTA
DEP'T of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
CONSOLIDATION DATA

PROJECT	Muskeg Research
SITE	W of Alsike
SAMPLE	Shelby Tube - A
LOCATION	1+83 NE 4' R of ℄
HOLE	#2
DEPTH	1'0"
TECHNICIAN	Koa
DATE	Dec 31/58

Ring Data

Ring No. 0

Weight gm's. 1368.0

Thickness ins. 1.47

Diameter ins. 2.60 in.

Area sq. cm's 34.3

Machine Data

Machine No. 5

Multiplication Factor 100

Wt. Block + Stone + Ball gm's. 377.8

Consolidation Sample Weights

Wt. Tare + Ring + Soil + Water (End) gms 1803.1

Wt. Tare + Ring + Soil (End) gms. 1755.1

Wt. Tare (Tare No. 2) gms. 375.5

Wt. Ring + Soil + Water (End) gms. 1427.6

Wt. Ring + Soil + Water (Start) gms. 1459.1

Wt. Ring + Soil gms. 1379.6

Wt. Soil gms. 11.6

Water (End) = 48.0 gms. = 414 %

Water (Start) = 79.5 gms. = 685 %

Description of Sample: Fibrous Muskeg - FI type not decomposed.

Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks
Dec 31 10 gm. (0.04 T/ft) 11:36 0 - 6900 6 - 6750 15 - 6740 30 - 6725 1m - 6715 2 - 6693 Add water. 11:54 - 6595 30 gm (0.10 T/ft) 11:51 0 - 6595 6 - 6140 15 - 6100 30 - 6040 1m - 6010 2m - 5970 4 - 5940 8 - 5880 16 - 5825 97 - 5730 143 - 5693 1560 - 5440 (Jan 1 1:51) Jan 2 4:50 AM - 5275 2860 (46h)	Jan 2 (0.16 T/ft) 50 gm 10:30 0 - 5275 6 - 5010 15 - 4950 30 - 4920 1m - 4880 2 - 4850 4 - 4820 9 - 4765 16 - 4720 34 - 4654 68 - 4594 101 - 4554 175 - 4505 300 - 4220 Jan 3 9:30 AM 1380 - 3940 1470 - 3900 Jan 4 1:30 3060 - 3780 Jan 5 1:30 4500 - 3680 Jan 6 11:30 5820 - 3620 97 hr 00'	Jan 6 80 gm (0.25 T/ft) 11:45 0 - 3620 6 - 3400 15 - 3360 30 - 3320 1m - 3310 2m - 3280 4 - 3250 8 - 3210 16 - 3165 35 - 3110 142 - 3020 250 - 2930 1810 - 2705 Jan 8 3060 - 2610 Jan 9 4:25 4620 - 2534 Jan 10 9:45 5640 - 2470 94 hrs 00	Jan 10 160 gm (0.50 T/ft) 10:30 0 - 2470 6 - 2150 15 - 2000 30 - 1910 1 - 1840 2 - 1790 4 - 1715 8 - 1640 16 - 1525 43 - 1305 94 - 1180 Jan 11 11:15 1500 - 0970 Jan 12 9:00 AM 2800 - 0815 Jan 13 11:30 4380 - 0658 Jan 14 1:45 - 0628 99 hr 15 m	Jan 14 340 gm (1.1 T/ft) 1:45 Move dial 0628 = 4628 0 - 4628 6 - 4300 15 - 4280 30 - 4240 1 - 4180 2 - 4105 5 - 4020 8 - 3970 90 - 3810 210 - 3680 Jan 15 9:45 1200 - 3460 Jan 16 8:45 2580 - 3340 43 hr 00 m	Jan 16 680 (2.0 T/ft) 9:50 AM 0 - 3340 6 - 3100 15 - 3010 30 - 2930 1 - 2860 2 - 2790 4 - 2725 8 - 2650 16 - 2570 30 - 2485 97 - 2390 140 - 2390 240 - 2330 330 - 2330 Jan 17 4:45 1420 - 2160 Jan 19 11:00 4380 - 2045 73 hrs 10 m. Rebound Jan 19 30 gm 9:00 0 - 2045 Jan 20. 1420 - 3200 24 h 00 m	Jan 20 10 gm. 9:00 AM. 0 - 3200 Jan 21 4:00 m. 1420 - 3640 24 h.

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
CONSOLIDATION RESULTS

PROJECT Muskeg Research	
SITE W of Alsike	
SAMPLE Shelby Tube – A	
LOCATION 1+83 NE	4' R of ℄
HOLE #2	DEPTH 1'-0"
TECHNICIAN Koa	DATE Dec 31/58

Assumed Specific Gravity of Soil Solids G_s = 1.40 Height of Soil Solids H_s = 0.095 ins.

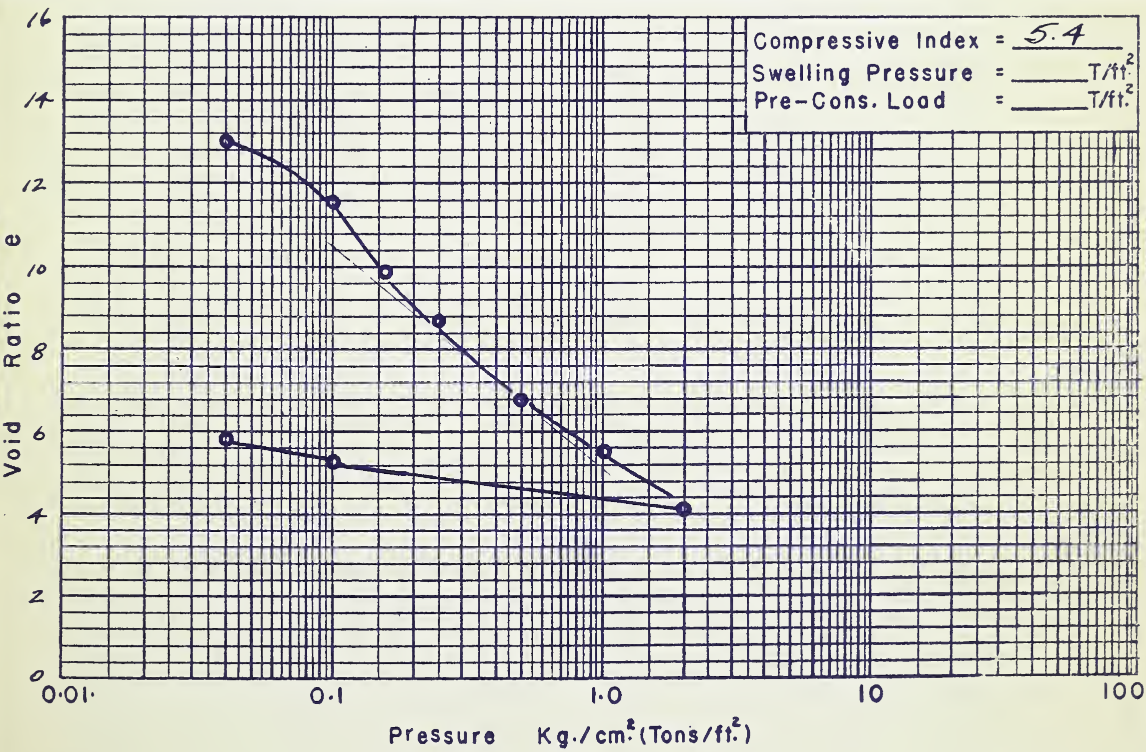
Void Ratio e (End) = 5.8

Void Ratio e(Start) = 9.6 (not saturated)

Void Ratio e (Start Dimensions) = 13.5

$e(\text{End}) = W\%(\text{End}) \times G_s$ $H_s = \left(\frac{\text{Wt. Soil}}{G_s \times \text{Area} \times 2.54}\right)$ ins. $e = \text{previous } e \pm \frac{\text{Def'l.}}{H_s}$

Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	Deflection / H_s	Void Ratio e	Pressure Kg/cm² = T/ft²
24 hr	10	3640	—	—	5.8	0.04 rebound
24 hr.	30	3200	-0.0440	-0.5	5.3	0.10 rebound
73 h. 10 m	680	2045	-.1155	-1.2	4.1	2.0
43 h. 00	340	3340	+1295	+1.4	5.5	1.1
99 h. 15 m	160	0628 = 4628	+1288	+1.3	6.8	0.50
94 h 00	80	2470	+1842	+1.9	8.7	0.25
97 h 00	50	3620	+1150	+1.2	9.9	0.16
46 h 00	30	5275	+1655	+1.7	11.6	0.10
15 m	10	6695	+1320	+1.4	13.0	0.04



Compressive Index = 5.4
Swelling Pressure = ______ T/ft²
Pre-Cons. Load = ______ T/ft²

UNIVERSITY of ALBERTA
DEP'T of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

CONSOLIDATION DATA

PROJECT: Muskeg Research
SITE: W of Assike
SAMPLE: Shelby Tube - B
LOCATION: 1+83 NE 4' R of ℄
HOLE: #2 DEPTH: 11'-0"
TECHNICIAN: Koa DATE: Dec 30/58

Ring Data
Ring No.: C-3
Weight gm's.: 903.0
Thickness ins.: 1.50
Diameter ins.: 2.60
Area sq. cm's.: 34.3

Machine Data
Machine No.: 4
Multiplication Factor: 100
Wt. Block + Stone + Ball gm's.: 365.74

Consolidation Sample Weights
Wt. Tare + Ring + Soil + Water (End) gms: 1220.8
Wt. Tare + Ring + Soil (End) gms.: 1138.5
Wt. Tare (Tare No. 1) gms: 222.90
Wt. Ring + Soil + Water (End) gms.: 997.9
Wt. Ring + Soil + Water (Start) gms.: 1020.5
Wt. Ring + Soil gms.: 915.6
Wt. Soil gms.: 12.6
Water (End) = 82.3 gms. = 656 %
Water (Start) = 104.9 gms. = 830 %

Description of Sample: Dark brown - black (Peat) Muskeg - well decomposed

Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks	Date, Time Load, Dial Remarks
Dec 30	80 gm (0.25 T/ft)	Jan 2	Jan 6	Jan 10	Rebound	
10 gm (0.04 T/ft)	11:49	160 gm (0.50 T/ft)	340 gm (1 T/ft)	680 gm (2.0 T/ft)	50 gm.	
11:00	0 - 4952	0 - 3350	11:25	10:35	Jan 14	
0 - 5885	7 - 4765	6 - 3015	0 - 1730	0 - 3000	1:45 - 3050	
10 - 5850	15 - 4755	15 - 3010	6 - 1520	6 - 2870	Jan 14	
30 - 5850	30 - 4550	30 - 3005	15 - 1450	15 - 2850	10 gm	
1 - 5850	1 - 4300	1 - 3000	30 - 1390	30 - 2825	Jan 16:	
2 - 5850	2 - 4280	2 - 2980	1 - 1330	1 - 2815	9:30 - 3490	
Add water	4 - 4250	4 - 2965	2 - 1260	2 - 2800	1:15 3620	
14 - 5832	8 - 4215	8 - 2935	4 - 1200	4 - 2785	1:45 - 3650	
30 gm (0.10 T/ft)	16 - 4162	16 - 2740	8 - 1120	8 - 2765	Jan 17	
11:15	35 - 4136	30 - 2520	16 - 1015	32 - 2750	3780	
0 - 5832	90 - 4080	60 - 2430	42 - 0850	82 - 2735		
10 - 5750	240 - 4000	85 - 2404	142 - 0725	Jan 11		
20 - 5750	290 - 3957	160 - 2348	250 - 0650	11:15		
40 - 5749	Dec 31	260 - 2290	Jan 7	1500 - 2680		
1 - 5749	9:45 - 3876	700 - 2200	10:45	Jan 12		
2 - 5748	11:45 - 3800	Jan 3	1410 - 0475	9:00		
4 - 5747	(1440)	9:30	5:30	2800 - 2665		
8 - 5747	1:25 - 3720	1380 - 2125	1810 - 0340	Jan 13		
50 gm (0.16 T/ft)	Jan 1	Jan 4	Jan 8	11:00		
11:30	1:25	1:30	2:35	4380 - 2620		
0 - 5746	2980 - 3520	3060 - 2015	3060 - 0225	frame on dish.		
30 - 5100	9:50	Jan 5	Jan 9			
1 - 5032	4180 - 3350	4500 - 1880	4:55 - 0155			
2 - 4995	70h - 01 m	Jan 6	Jan 10			
4 - 4977		5820 - 1730	10:20			
8 - 4964		1:30	5640 - 0040			
16 - 4952		97h 0m	94 hr 55 m.			
			Moved dial			
			0040 = 3000			

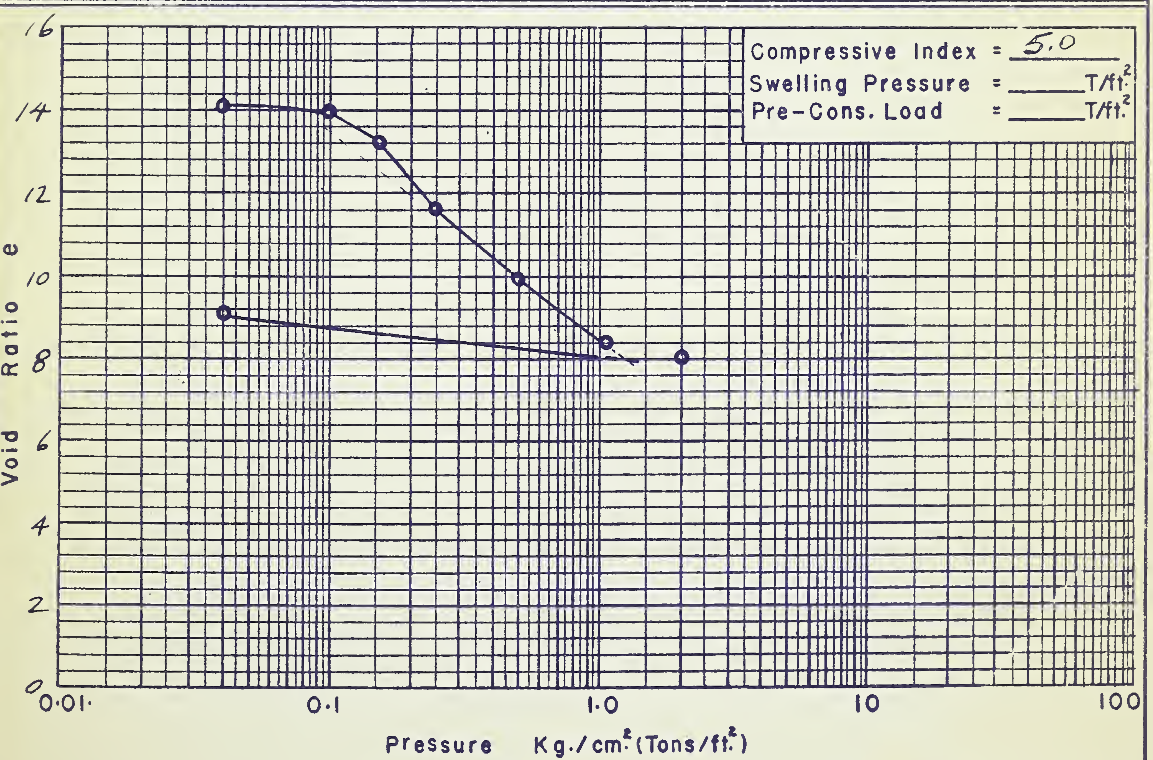
UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
CONSOLIDATION RESULTS

PROJECT Muskeg Research
SITE Kl of Alsike
SAMPLE Shelby Tube - B
LOCATION 1+83NE 4'Rt of ℄
HOLE #2 DEPTH 11'-0'
TECHNICIAN Log DATE Dec 30/58

Assumed
Specific Gravity of Soil Solids G_s = 1.40 Height of Soil Solids H_s = 0.103 ins.
Void Ratio e (End) = 9.1
Void Ratio e(Start) = 11.6 (Not Saturated)
Void Ratio e (Start Dimensions) = 14.2

$e(End) = W\%(End) \times G_s$ $H_s = \left(\frac{Wt \cdot Soil}{G_s \times Area \times 2 \cdot 54}\right)$ ins. $e = \text{previous } e \pm \frac{Def'l.}{H_s}$

Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	Deflection / H_s	Void Ratio e	Pressure Kg/cm² = T/ft²
24h 00	10	3780	—	—	9.1	0.04 (Rebound)
98h 25m	680	2620	−0.1160	−1.1	8.0	2.0
94h 55m	340	3000 = 0040	+0.038	+0.4	8.4	1.1
97h 0m	160	1730	+0.1690	1.6	10.0	0.50
70h 01m	80	3330	+0.1600	1.6	11.6	0.25
16m	50	4952	+0.1622	1.6	13.2	0.16
8m	30	5747	+0.0795	0.8	14.0	0.10
14m	10	5832	+0.0085	+0.1	14.1	0.04



Compressive Index = 5.0
Swelling Pressure = ______ T/ft²
Pre-Cons. Load = ______ T/ft²